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WATER POLLUTION CONTROL FEDERATION

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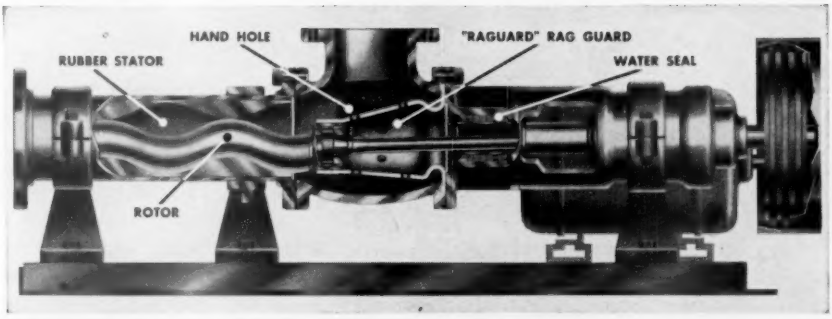

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Volume 32 | February 1960 | Number 2

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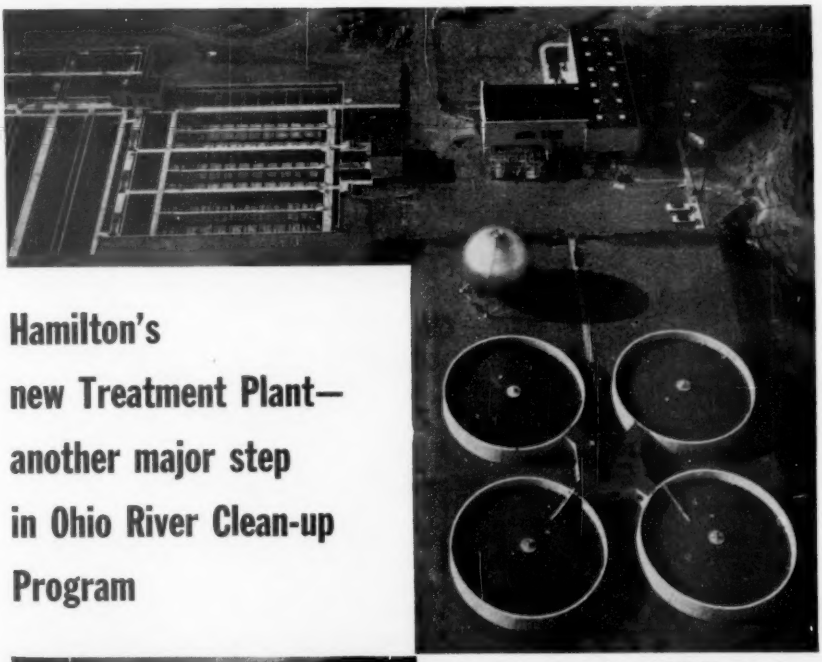

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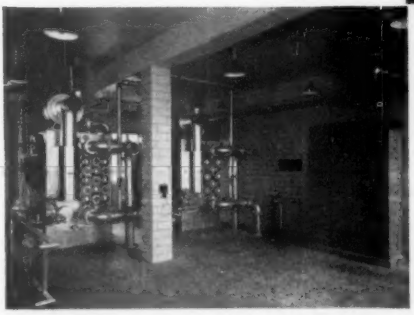

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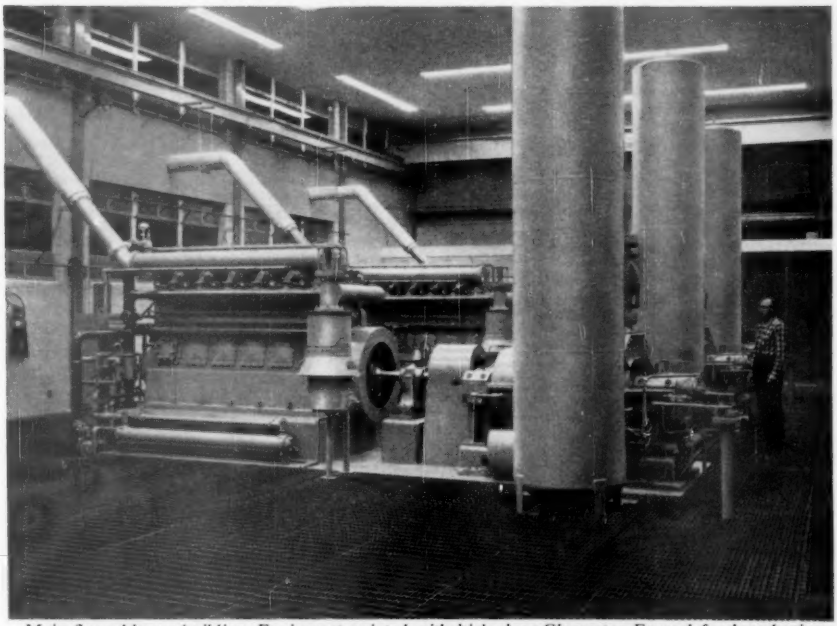

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Arkansas Water and Sewage Conf.	Marion Hotel Little Rock, Ark.	Mar. 13–16
Louisiana Conference on Water Supply and Sewerage	La. State Univ. Baton Rouge, La.	Mar. 16–18
West Virginia Water Pollution Control Assn.	Hotel Pritchard Huntington, W. Va.	Mar. 23–24
New Jersey Sewage and Industrial Wastes Assn.	Traymore Hotel Atlantic City, N. J.	Mar. 30–Apr. 1
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Montana Sewage and Industrial Wastes Assn.	Northern Hotel Billings, Mont.	Apr. 28–30
Michigan Sewage and Industrial Wastes Assn.	Park Place Hotel Traverse City, Mich.	May 23–25
Föreningen för Vattenhygien (Sweden)	Institutet för Arbetshygien Helsinki, Finland	May 27–29

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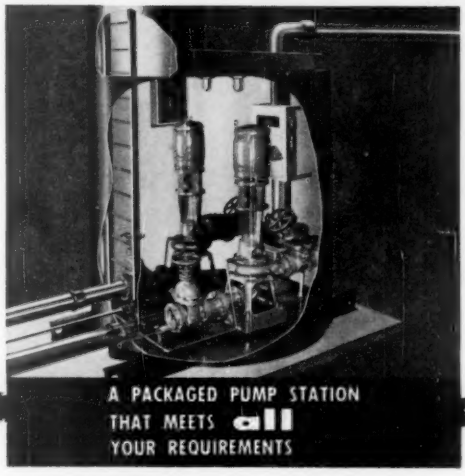

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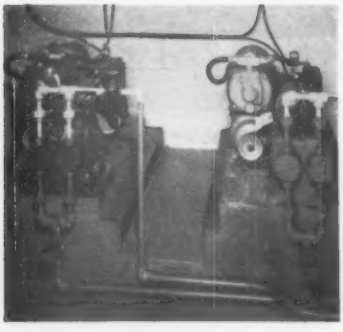

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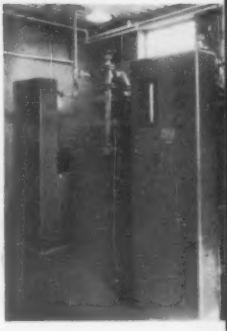

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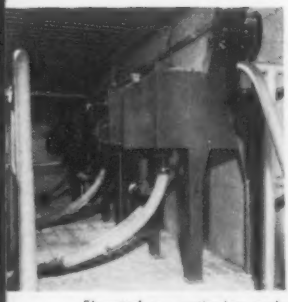
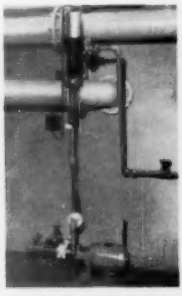
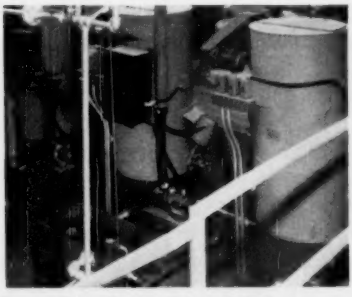
Investigate the complete B-I-F line of waste treatment equipment. Contact **B-I-F Industries, Inc., 368 Harris Ave., Providence 1, R. I.**



Elevated proportioning weir tanks, automatically or manually controlled, feed lime slurry directly to points of application.



pH control system automatically maintains optimum pH for chemical treatment. Continuous representative sample pumped through "flow-through" pH electrode assembly produces pneumatic signal controlling chemical feed.



Three Omega Universal feeders, accurately feed 16 to 65 pounds per hour of dry chemical, permit manual setting of feed rate over extra wide range for treating varying strength wastes.

"Not a 'comeback' in 18 years" ...with Transite Sewer Pipe

says Anthony Garcia, President
Allerton Construction Co., New York

"In my eighteen years of installing Transite sewer mains and house laterals, I have never had a 'comeback' because of line failures. Most of the installations have been 100% under water. Therefore, tight joints that stay tight are absolutely essential! For these reasons Transite has always been my choice for sewer line service. I never worry about passing leakage or infiltration tests. I play it safe—with Transite."

You'll want to read how Johns-Manville Transite® Pipe has helped other contractors increase their profits and productivity . . . in the informative and interesting new, 8-page booklet, TR-206A. Send for it now! Address Johns-Manville, Box 14, New York 16, New York. In Canada, Port Credit, Ontario.

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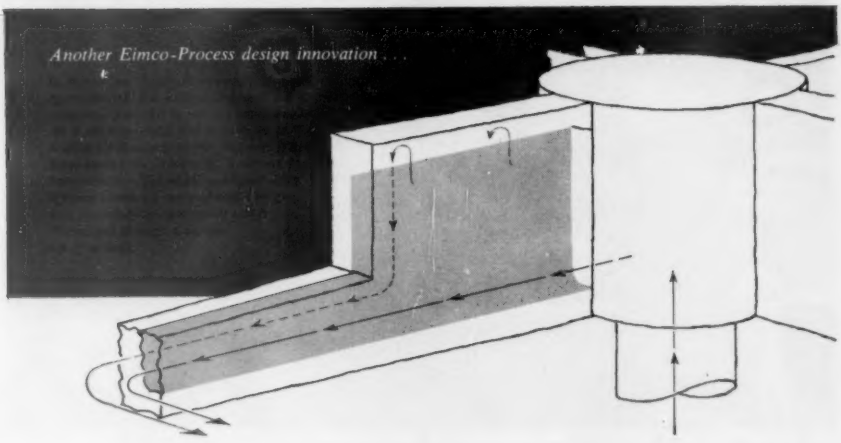

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Two common problems in high rate biological filtration are widely varying flow rates and low available hydraulic head. Both are solved by this exclusive Eimco-Process design that provides external radial flow division sections and compartmented arms to efficiently handle both high and low flows at a relatively constant low head. The design is suitable for either two or four arm construction with all arms discharging at all times.

This is only one of the superior Eimco-Process Rotary Distributor types available for all bed sizes and for influent column diameters ranging from 8 to 60 inches. Mechanisms for both standard and high rate filtration are included, with or without internal flow control provisions, with pipe or rectangular steel plate arms, and with a choice of seals. For recommendations on your application, please contact any of our nationwide sales offices.

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Process Engineers, Inc. Division
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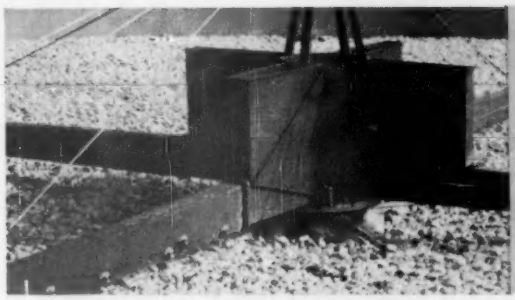
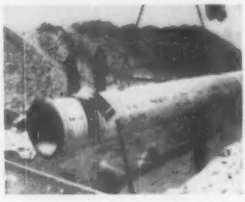


. . . another outstanding installation

Radial weir center assembly on a 102 ft. dia. high rate Rotary Distributor installed at the Arlington, Texas, Sewage Treatment Plant. Consulting Engineers—Reaves and Gregory, Fort Worth, Texas. All types of Eimco-Process Rotary Distributors are described in our new bulletin SM-1011 . . . a copy is yours on request

Twenty-foot lengths of Armco SMOOTH-FLO Pipe helped maintain uniform grade.

How Garden City, Kansas, installed a sewer in soupy soil



Water in the trench didn't stop installation of Armco Pipe.

Garden City, Kansas, recently installed a 1-mile interceptor sewer to feed their new treatment plant. Part of this line passed through extremely wet soil.

The contractor installed some 5,800 feet of Armco SMOOTH-FLO® Sewer Pipe in 20-foot lengths, with positive beam strength to maintain a continuous uniform grade in this unstable soil. SMOOTH-FLO's dense interior lining provided the necessary high flow capacity. An alternate material specification, for short-section pipe with nonpositive joints, called for expensive rigid bedding to maintain grade. The strong, lightweight Armco Pipe went in quickly and economically — with no special bedding required.

Get complete details on Armco Corrugated Metal Pipe. Write for catalog, "Armco Sewer Structures." Armco Drainage & Metal Products, Inc., 4350 Curtis Street, Middletown, Ohio.

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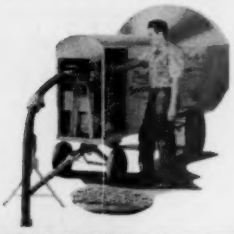
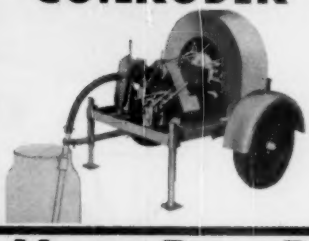


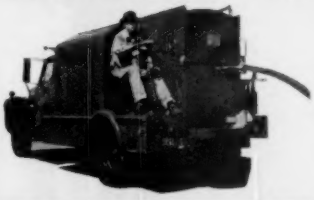
TRUCK-LODER

COILRODER



BUCKET MACHINE

MONORODER



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The "boys" are back . . . !
by popular demand!
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Tyton Joint® pipe is *almost* as easy to install as our hillbilly friend indicates. Only one accessory needed . . . a specially designed rubber gasket that fits into the bell of the receiving pipe. A push or two and the connecting pipe compresses the gasket . . . seals the joint bottle-tight and permanently.

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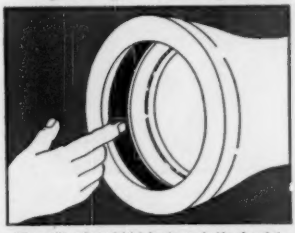
A WHOLLY INTEGRATED PRODUCER FROM MINES AND BLAST FURNACES TO FINISHED PIPE

INDUSTRIAL SERVICE CAST IRON

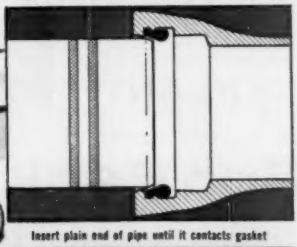
TYTON

ONLY FOUR SIMPLE ACTIONS

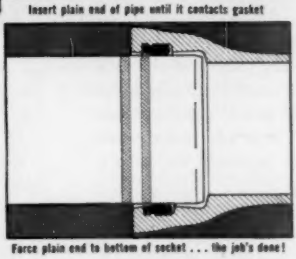
Insert gasket with groove over bead in gasket seat



Wipe a film of special lubricant over inside of gasket



Insert plain end of pipe until it contacts gasket



Force plain end to bottom of socket . . . the job's done!

MODERN AS TOMORROW

Plant Superintendent: Harold W. Augenstein
Consulting Engineers: Consoer, Townsend & Associates, Chicago

Primary tanks with Rex Conveyor Sludge Collectors and Roto-Skim Pipes.

Final tanks with Rex Unitube Tow-Bro Sludge Removers.

HAMILTON, OHIO, PLANT

Planned For Peak Performance

The Hamilton, Ohio, new split activated sludge sewage treatment plant is a tribute to sound planning that considered not only today's but tomorrow's requirements. In anticipation of a substantial population and industrial expansion, the planning team designed a plant that would provide maximum efficiency with adequate reserve capacity for future growth of the community.

The new plant has a designed average flow of 12 m.g.d., with a maximum designed flow of 18 m.g.d. handling both municipal and industrial wastes. In the design of the Hamilton plant, maximum utilization of available area was accomplished through a compact arrangement of the treatment units by using common wall construction throughout.

The grit chambers, equipped with Rex Chain and Bucket Grit Collectors, and the rectangular primary tanks, with Rex Conveyor Sludge Collectors and Roto-Skim Pipes, are neatly grouped with the rectangular aeration tanks and the three square final tanks. Future expansion of these units with a minimum of cost is guaranteed.

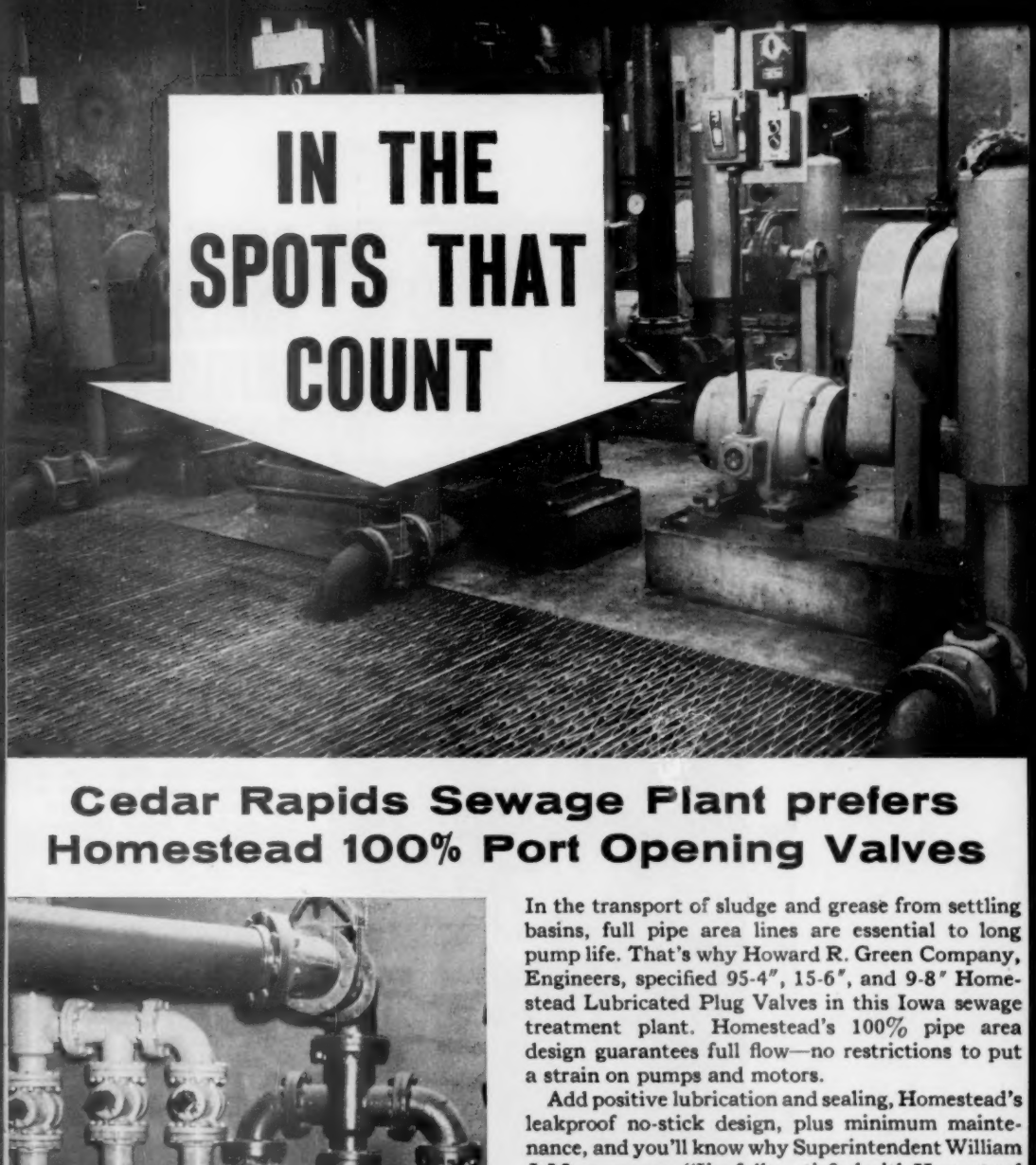
In the final tanks, Rex Unitube Tow-Bro Sludge Removers assure maximum flexibility in operation ...both today and for years to come. With more positive and general removal of sludge, the Tow-Bro provides ample facilities for efficient operation under varied conditions.

Rex Waste Treatment Engineers are available to help design and select equipment for modern plant performance as at the Hamilton, Ohio, plant. For complete information, write CHAIN Belt Company, 4606 W. Greenfield Ave., Milwaukee 1, Wisconsin.

REX®

CHAIN BELT COMPANY

Specialized Equipment for the Effective Treatment of Water, Municipal and Industrial Wastes.



IN THE SPOTS THAT COUNT

Cedar Rapids Sewage Plant prefers Homestead 100% Port Opening Valves

Homestead 100% Port opening Lubricated Valves controlling supernatant, sludge and gas in Cedar Rapids Sewage Plant.

In the transport of sludge and grease from settling basins, full pipe area lines are essential to long pump life. That's why Howard R. Green Company, Engineers, specified 95-4″, 15-6″, and 9-8″ Homestead Lubricated Plug Valves in this Iowa sewage treatment plant. Homestead's 100% pipe area design guarantees full flow—no restrictions to put a strain on pumps and motors.

Add positive lubrication and sealing, Homestead's leakproof no-stick design, plus minimum maintenance, and you'll know why Superintendent William J. Meaney says, "I'm fully satisfied with Homestead Valves on our lines."

Mail coupon today for Reference Books 39-1 and 1A—a complete library of Homestead Lubricated Plug Valves and their uses.

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DON'T BE FOOLED BY INITIAL PIPE COSTS...

Certain factors concerning the type of pipe to be used for a proposed water or gas project must be examined carefully:

First — how much does the pipe cost, compared to other types?

Second — how often will it require repair?

Third — how long before it has to be replaced?

Sometimes the first cost of jobs where cast iron pipe is specified, is higher than similar projects using cheaper pipe. Yet, in the long run, cast iron pipe costs *less!* Here's why:

- Cast iron pipe rarely requires repairs. Its rugged construction, corrosion-resistant qualities and bottle-tight rubber-ring joints will withstand the most severe pressures.

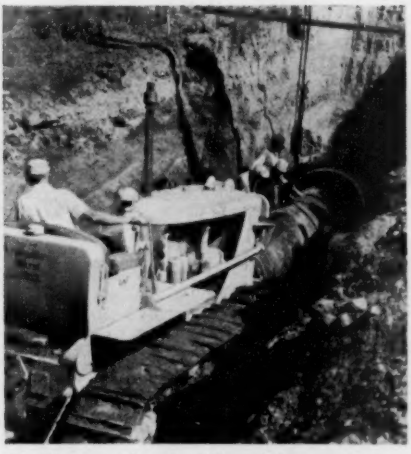
- Cast iron pipe is built to *last*—98 American cities will testify to that. They've had cast iron pipe installations in constant use for over a century! Once cast iron pipe is in the ground, it stays there!

Don't be fooled by "low-cost" pipe. Insist on the pipe that will actually save you money over a period of years.

In Nebraska—Here a section of cast iron pipe is being relocated. Twenty-five years old, the pipe is still in excellent condition—has never required major repair . . . or replacement.

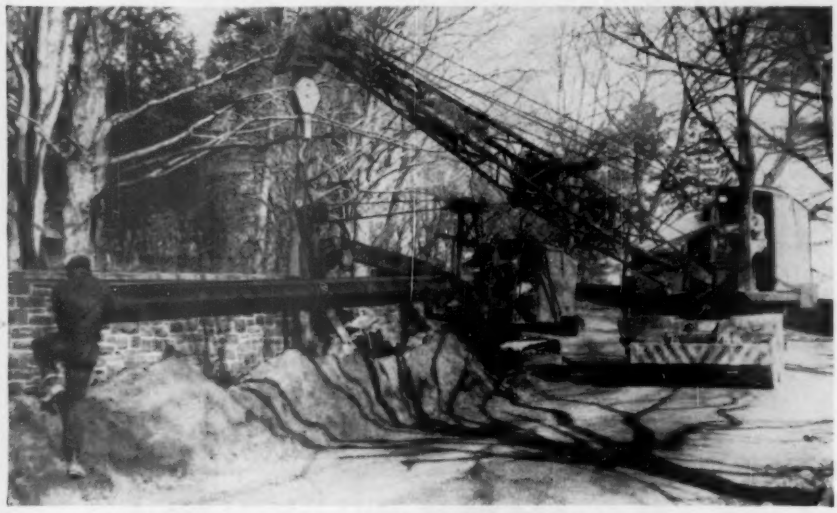
Rely on CAST IRON PIPE

In Indiana (above)—"All-weather" cast iron pipe is quickly installed despite wet trench conditions. Slip-on joints were easily assembled, with one workman using a crowbar.



In Kansas (above)—36" cast iron water main. Another part of this main was floated out of its trench by a heavy downpour. Later a 150 psi water test revealed no leaks in the slip-on joints!

In Pennsylvania (below)—This 16" cast iron pipe is being installed as fast as the trench hoe can prepare the trench. Handy lengths and slip-on joints make cast iron pipe easy to handle, even in crowded neighborhood sectors; require less labor.



CAST IRON PIPE
THE MARK OF THE 100-YEAR PIPE
Cast Iron Pipe Research Association, Thos. F. Wolfe
Managing Director, 3440 Prudential Plaza, Chicago 1, Ill.

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IOWA
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Iowa's complete line of valves for treatment plants assures you of efficient operation, low maintenance cost, and replacement parts when needed in years to come.

IOWA CHECK VALVES—Swing gate type for use in either horizontal or vertical pipe lines. Bronze trimmed throughout. Stainless steel hinge pins. Leather or rubber faced gates, or solid bronze, as desired—aluminum gates available for air service.

IOWA MEDIUM AND LOW PRESSURE VALVES—either non-rising stem or outside screw and yoke types. Designed as carefully as those conforming to A.W.W.A. specifications but with lighter construction for lower pressures.

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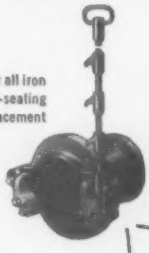
IOWA PLUG DRAIN VALVES—either rising or non-rising stem type. High strength cast iron bodies; bronze stem, operating nut, disc ring and seat ring; rustproof steel bolts and nuts. Handwheels, extension stems, plain or indicating floor stands if required.

IOWA SLUICE GATES—can be used in lines having both seating and unseating pressures. High strength cast iron, bronze mounted, with solid bronze adjustable wedges. Can be equipped with cylinder or motor unit for automatic operation.

IOWA SHEAR GATES—either all iron or iron, bronze mounted. Gate-seating wedges bolted on, permitting replacement without replacing frame.



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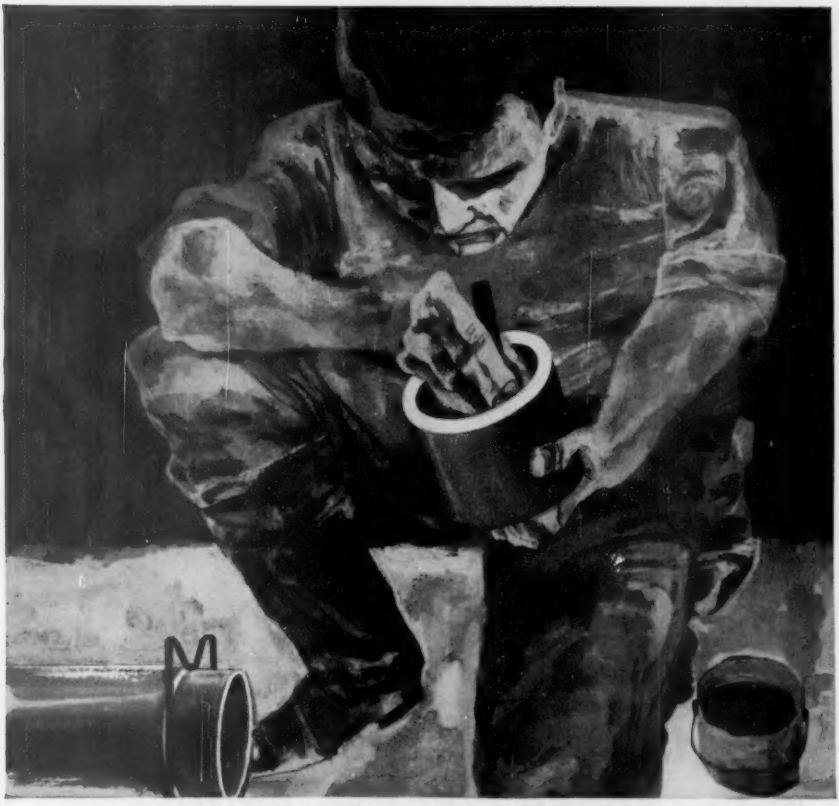

Exclusive FLUID-TITE Coupling joins lengths of varying crushing strengths and sizes. Assemble in any weather—without heavy machinery.

"K&M" ASBESTOS-CEMENT SEWER PIPE

Infiltration-proof at 25 psi pressure!

Exclusive, patented "K&M" FLUID-TITE Coupling maintains a permanently infiltration-proof seal under severest field conditions.

This is true even when "K&M" Asbestos-Cement "FLUID-TITE" Sewer Pipe is under deflection, or is buried at various depths under heavy backfill. Tests prove this infiltration-proof pipe can withstand 25 psi external water pressure.

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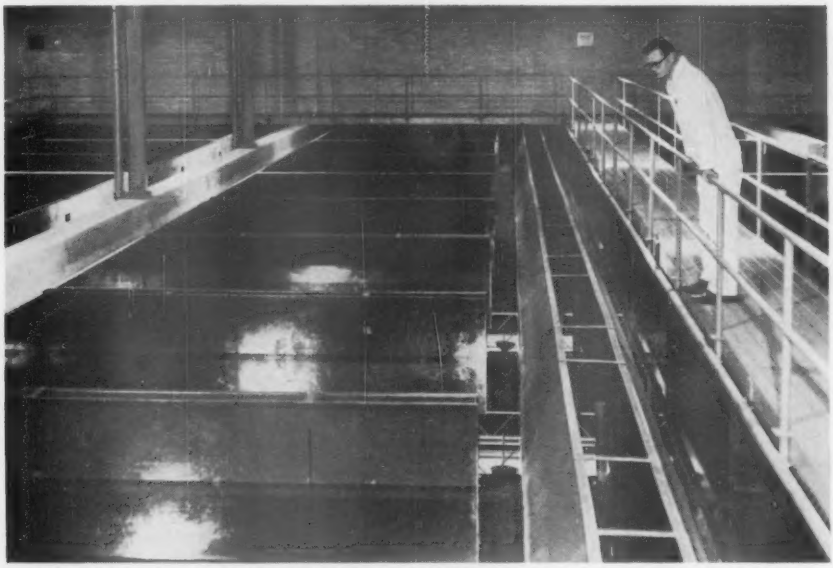

SPACE SAVER. Permutit Horizontal Precipitators take only 25% as much space as would be needed by conventional basins at the Town of Tonawanda Water Treatment Plant, Tonawanda, N. Y.

How to get the capacity you need in the space you can afford

When the booming town of Tonawanda, N.Y., decided to build its own water treatment plant, space was a major consideration.

The solution as proposed by Nussbaumer, Clarke and Velzey, Consulting Engineers, was to install four Permutit Horizontal Precipitators.

Each unit measures 31' 6" x 95' x 16' 3" and is designed to handle four million gallons per day with a two million gallon overload. Proved in operation for more than two years, this system efficiently treats 24 million gallons per day.

And this huge load is handled using only one-fourth the space that would have been required if conventional basins had been used.

Space is just one saving you can make with Permutit Precipitators. You'll also find that treatment efficiency is increased, and that often there's a reduction in chemicals and coagulants required.

If you'd like more information about the efficiencies possible through the use of Permutit Precipitators, write for Bulletin No. 2204C.

Or, for a brief introduction to the full line of Pfaudler Permutit equipment for water and waste treatment, ask for the bulletin entitled "An Outline of Modern Water Treating Equipment." Address your inquiries to our Permutit Division, Dept. SI-20, 50 West 44th Street, New York 36, New York.

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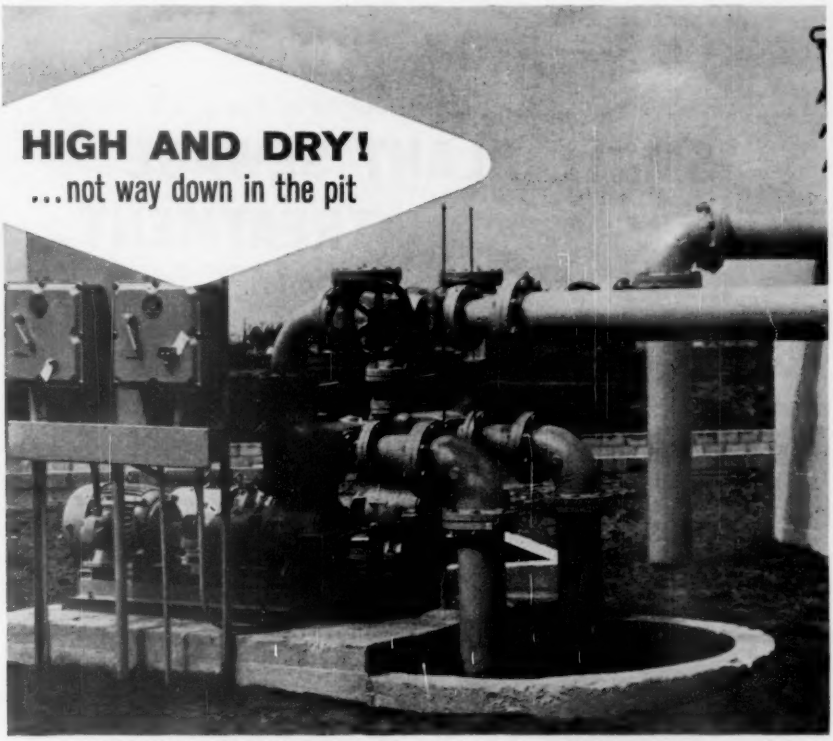

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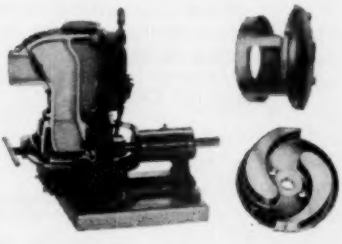
GORMAN-RUPP SEWAGE PUMPS

COST LESS TO BUY—Self-priming in lifts up to 15', positive in action, nearly completely non-clogging. High and dry, their installed cost has allowed savings of as much as 50%.

COST LESS TO MAINTAIN—Almost no maintenance needed! No need to service by entering pit or raising pump. Just a turn of the wrist and the end plate is off.

PERFORM WHERE OTHERS FAIL — In lift stations and sewage plants, these Gorman-Rupp units have shown amazing records of savings . . . operating where others have failed. Let us show you examples.

2", 3", 4" and 6" pumps—connected to your power or ordered complete with power units. Write for complete specifications.



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SMALL PLANT SEWAGE TREATMENT

Just published! A brand-new bulletin on revolutionary new developments in low-cost plants for communities up to 10,000

Sewage treatment plants, with all essential operations performed mechanically and continuously in single, low-cost combination units, now bring the advantages of modern sewage handling methods within reach of even the smallest community.

With only one tank and one drive unit to operate, these new plants overcome the problem of high costs which often make conventional treatment facilities beyond the means of small users. Latest improvements include mechanical grit removal combined with sludge digestion and clarification.

If you're interested in small plant sewage treatment for communities up to 10,000 or even for individual industrial plants, schools, motels or housing developments, you'll want the new bulletin shown above – just off the press! Write for Bulletin No. 6692 to Dorr-Oliver Incorporated, Stamford, Connecticut.

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WORLD-WIDE RESEARCH • ENGINEERING • EQUIPMENT

Sewage Works

TECHNOLOGY AND ECONOMICS OF HOUSEHOLD SEWAGE DISPOSAL SYSTEMS

By Harold A. Thomas, Jr., James B. Coulter, Thomas W. Bendixen, and Allan B. Edwards

Respectively, Professor of Civil and Sanitary Engineering, Harvard University, Cambridge, Mass.; Senior Sanitary Engineer, Soil Scientist and Senior Assistant Sanitary Engineer, Suburban Sanitation Studies, Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio

This paper was a report to the Committee on Sanitary Engineering and Environment and the Subcommittee on Waste Disposal of the National Research Council, Division of Medical Science; approved and released for publication Dec. 2, 1959. The report was titled: "Individual Household Sewage Disposal Systems: Developing Technology, Cost Analyses, and Research Needs."

Introduction

In the technical literature of the social sciences during the postwar years much attention has been given to the development of the new exurban pattern of living in the United States. An expansion and diffusion of population into and beyond the peripheral zones of metropolitan areas has occurred at an increasing rate as a result of the increased utility of the automobile and investment in highways. The new Federal highway program may be expected to accelerate the growth of "ribbon" cities and the diffuse "wheel spoke" structure of metropolitan areas. There is little doubt but that the trend will continue in the foreseeable future.

The movement of population has created new problems for the sanitary engineering profession. Among these the most demanding are the problems of improving present systems for the disposal of liquid wastes from individual households and small communities and in developing new methods.

Coulter *et al.* (1) presented an analysis of the current changes in population distribution based on census figures and other sources of statistical information. From this analysis it is concluded that "The dwindling farm population, the demand for home-ownership, modern home financing, scarcity of land served by sewers, expressways, and a phenomenal increase in the birth rate are all factors indicating that the problem of sewage disposal in suburban areas will become increasingly acute for many years to come. . . . Economic analysis and metropolitan planning must (increasingly) become . . . standard practice for sanitary engineers if costly and aggravating errors are to be avoided."

Following the "sanitary revolution" of the nineteenth century the sanitary engineer emerged with two techniques for the disposal of human wastes that have stood well the test of time. In areas of high population density the water-carriage system of disposal has proved to be effective and economical;

in rural areas the sanitary disposal of excreta without water-carriage by methods such as the pit privy have proved to be satisfactory. However, between these two extremes, in regions of intermediate population density including the new exurban communities and diffuse fringe areas, it has often been difficult and sometimes impossible to provide a satisfactory sewage disposal system. In these areas the cost of sanitary sewerage may, during the initial period of growth, be high to the point of being prohibitive. On the other hand the population density and the standard of living in these places are often too high to be satisfied by excreta disposal without water-carriage.

Septic Tank Usage

In regions where construction of community sewage collection and treatment systems is not immediately practicable for financial reasons, the most widely used sewage treatment process for individual households has been treatment of water-carried wastes in septic tanks coupled with soil absorption systems. This process and related systems have been and will continue to be satisfactory in areas with suitable types of soil, topography, and other favorable factors. However, there are large sections of the country with impervious soils or otherwise unsuitable conditions where the method has not been successful. Dissatisfaction with the septic tank-tile field method and with related methods in their present state of development exists among local, state, and federal health and water resource regulatory organizations. The dissatisfaction with the septic tank method derives from four sources:

1. Maintenance by the householder is often erratic or completely lacking. Adequate inspection and administration of control by health organizations is difficult and expensive. When sludge is not removed at proper intervals from the septic tank it will be discharged together with some fresh sewage solids in the overflow and impair or destroy the receiving capacity of the soil absorption system—even if the latter is installed in sand with a high capacity for absorbing normal effluent. When the absorption system fails, raw or partially treated sewage seeps onto the ground surface and may become a nuisance and a public health hazard.

2. All soil, except the most pervious formations, will become clogged with the solid organic and inorganic residues of stabilized sewage. Most absorption systems must be revamped or replaced at more or less regular intervals depending on the porosity and permeability of the soil, the organic loading, the area of the field, and the slope of the ground.

3. The present state of design as regards the absorption area requirement per person is not satisfactory. In some regions it is difficult to predict with precision the capacity of a given soil for receiving effluent. Soil types vary over a wide range. Designs based on fresh water percolation tests in current use are frequently wide of the mark. In many soils a low correlation exists between the observed short-time uptake capacity for fresh water as measured under unsteady state hydraulic conditions and the long-term, more or less steady state, percolation capacity for septic tank effluent.

4. Although when properly designed and installed, septic tank and soil absorption systems can give satisfactory service in areas where soil conditions are favorable, in actual practice most systems are installed without adequate attention to design and construction because of the high cost of adequate exploration of the soil and subsoil which may require soil borings as well as leaching tests. In these areas the inadequacy of septic tank systems may derive more from improper design and

poor construction than from inherent limitations of the device—in practice it has been difficult to assure that its full capabilities will be utilized.

As a consequence of these technical circumstances and the administrative problems they create, the use of the septic tank and absorption field system has resulted in unhygienic and uneconomical practices in many communities. The diffusion of a large segment of the population into zones of population density intermediate between urban and rural levels has intensified on nationwide basis the need for the development of safe and economical methods of sewage disposal for exurban and diffuse fringe area communities.

Water Conservation and Ground Water Pollution

In some regions the problem of water conservation is already acute. McKee (2) states "There is one facet of the problem that deserves specific attention, namely, the matter of wastewater reclamation through ground-water recharge. In southern California, for example, there are tremendous ground-water basins that serve as a part of the water supply for metropolitan areas. The ground-water table is normally several hundred feet below the ground surface. With individual household disposal systems, much of the domestic water is returned to the ground where it helps augment the natural sources of recharge. When a suburban area is sewered, on the other hand, the sewage is conveyed . . . to the ocean and is lost as a source of fresh water."

Another method of achieving water conservation is outlined in a subsequent section of this report in which aerobic treatment systems for household sewage are discussed.

Some authorities have stressed the lack of knowledge at present of factors governing the self purification of ground water and the persistence of ground-water contamination. A number of cases of well pollution by synthetic detergents from household sewage disposal systems have been reported by health organizations.

The foregoing considerations indicate the need for close coordination between research and development in sewage disposal methods and that relating to other segments of the water economy.

Classification of Types of Fringe Area Development

Conceptually it is useful to recognize that the radial movement of population from metropolitan centers may create two types of fringe areas: (*a*) those suburban zones, subdivisions, and shopping centers that will rapidly receive full urbanized development with high population density; and (*b*) those more diffuse and remote exurban areas where it appears that full development is out of the question for at least a generation. Much of the dissatisfaction with individual household treatment devices has resulted from experience in a large number of fringe areas of the first type where installation of these devices has proved to be a serious mistake. Not only have they failed to provide satisfactory sewage disposal, but more significantly they have retarded or even blocked the installation of conventional water-carriage systems, which ultimately for these areas provide the only safe and economic disposal method.

In the second type of development where it appears quite certain that the population density will remain at a low level for many years, there is no doubt but that the individual household disposal system has a useful and economically justifiable function.

In many potential fringe areas practicable difficulties arise of (*a*) predicting correctly whether the ultimate development will be heavy or light and (*b*) of foretelling when urbanization will be sufficiently complete to warrant

public sewerage. Moreover, around many cities there exists a nearly continuous gradient of population density and development level between fully urbanized and rural areas. Therefore, even though there were no uncertainties in predicting the time rate of urbanization, the administrator has the additional difficulty of identifying for a given place and time, the economic boundary zone delimiting the utility of community sewerage and that of the individual household disposal system.

A method of analysis of the economics and public health aspects of fringe area sewage disposal schemes is presented in a subsequent section of this paper. The method is intended to be useful in determining for specific sites the most economical resolution of the fringe area sewage disposal problem given reasonably satisfactory predictions of the ultimate level of development and the time rate at which this will be approached.

Classification of Channels for Research and Development

In laying the foundation for economic analysis it is pertinent to identify three distinct approaches to the needed improvement of the technology of sewage disposal for fringe areas.

1. The development of the technology of sewerage systems and community sewage treatment plants in the direction of reduced costs (initial and operational) and higher efficiencies. The emphasis here should be on the needs of the small community or isolated subdivision.

2. The development of new or modified *treatment* methods and devices, including those for disinfection, suitable for installation at individual houses.

3. The development of new or modified (ultimate) *disposal* methods and devices suitable for installation at individual houses. It is useful to recognize that in the absence of an adequate body of receiving water, there exist only three possibilities for the ultimate disposal of treated sewage effluents from individual households: the effluent can be discharged (wholly or in part) (*a*) into the soil where eventually it will join the ground water or be evaporated; (*b*) onto the ground surface and to natural drainage, if any; and (*c*) to the air by aerosol production, evaporation, and transpiration.

In assessing the relative merits of these approaches and in planning for future directed research and development it is pertinent to list and discuss briefly recent advances and trends in the technology of wastewater treatment and disposal.

Technological Developments

Sewerage

The principal limitation in the extension of the conventional water-carriage system to fringe areas has been its high cost. Many of the existing combined sewerage systems of large and medium-sized cities are already loaded to hydraulic capacity, and extension of these is not feasible without drastic and expensive construction. However, experience indicates that often an older system can accommodate considerable extension provided that extension is confined to sanitary sewage only.

New subdivisions and exurban communities are often located in areas where topographical conditions are unfavorable for gravity flow systems and for ultimate disposal in natural waterways of adequate receiving capacity. Moreover, the cost of sanitary sewerage is closely dependent on population density. The per capita cost in many situations varies approximately inversely as the square root of the areal density of population. According to this rule, the cost per person of sewerage in a zoning area with minimum lot

sizes of one acre is approximately three ($\sqrt{43{,}560/5{,}000}$) times that of zones with lots averaging 5,000 sq ft.

The Committee on Public Health Activities of the Sanitary Engineering Division of the American Society of Civil Engineers has recently presented an analysis of the various factors that operate to determine the zone of utility of the community sewerage system and that of the individual household disposal system (3). This report points out the poor economy that has occurred in many subdivisions where installation of individual systems has been permitted. In these places the construction of community sewers though requiring a greater amount of initial investment, would have resulted in substantial long-term saving.

The chief technological developments that tend to reduce sewerage costs are: (*a*) the continued improvement in the efficiency of trenching machinery and other types of earth-moving equipment; and (*b*) the utility of new materials such as plastics, for pipes. Along with these may be listed the steady improvement in efficiency and reliability of pumps, motors, and comminutors, and the reduced cost and increased availability of electric power.

An increasing number of engineers believe that these developments may be combined to make possible a new concept of separate sewerage for areas of intermediate population density with large lot sizes. The new systems would feature small diameter force mains of synthetic materials in relatively shallow trenches, having frequent curves, few manholes, and numerous small pumping and comminution stations.

A scheme developed in Sweden (4) incorporates some of these features; the sewers are under negative pressure for the vacuum transport of toilet wastes that have been separated from bath, dish, and laundry wastewater. The rinse water for toilet refuse moves as a piston-like plug through small plastic pipes which may be laid above or below the hydraulic grade line. A reduction in toilet water use of 60 to 80 per cent is claimed. The greatest economic gain comes from construction since less excavation and blasting are required. The Swedish Government has allocated state grants for continued development.

Small Community Sewage Treatment Plants

The principal improvement in sewage treatment for small communities has been in the direction of development of reliable components (pumps, motors, compressors, comminutors) and simplified processes that reduce operating and personnel costs. A particularly promising trend is the evolution of the "all aerobic" plant with elimination of settling and digestion units, and having only simple mechanical components such as pumps, comminutors, and compressors.

A number of equipment manufacturers with continued investment in development have perfected small "packaged" plants that are adaptations for the small community of conventional treatment methods such as the activated sludge and trickling filter processes. In many localities an administrative pattern has developed in which it is now possible for a single operator to maintain several of these small plants in adjacent communities.

There have been difficulties in some states in securing adequate operation of small plants, particularly in installations providing temporary treatment pending connection with larger metropolitan systems. Some health departments have therefore found it expedient to withhold permits for installation of packaged plants until there has been developed a master plan assuring adequate continuity and quality of operation, and providing for the elimination of these facilities in accordance with an accepted schedule for connec-

tion to the metropolitan sewerage system.

In this connection it is relevant to cite the increased recognition of the utility of sewage stabilization ponds for small communities in areas where climatic conditions are favorable. Experiments by Pasveer (5) have shown that the area of stabilization ponds may be reduced markedly when a small amount of mechanical aeration is introduced. Although there has been an encouraging amount of progress in establishing the capabilities of the stabilization pond, more work toward evaluation is desirable.

Individual Household Aerobic Sewage Treatment Systems

The trends and developments that have improved the small community plant have been extended and applied to the treatment and disposal of wastewater from individual households. The most promising result has been an adaptation of the activated sludge process. The development of household aerobic sewage treatment devices, which derives primarily from the work of Bloodgood (6), Perry (7), and Rigby (8), is an important forward step in sanitary engineering practice. Criteria for the design and the delimitation of the use of these devices have been reported by committee studies (9).

The new and essential technical innovation is the aerobic stabilization of sewage in aerated tanks in the presence of activated or semi-activated sludge for long periods—usually 24 hr or more. With the long period of oxidation the amount of sludge is greatly reduced. A small amount (normally) of residual spent excess sludge is carried out in the effluent. The elimination of the need for frequent sludge removal is an important achievement and ameliorates the maintenance problem. The practicality of some of the commercial and experimental systems has been enhanced by the development of ingenious devices for mechanical aeration and sludge return.

In one modification developed by Bloodgood (6), Perry (7), and Ribgy (8) water conservation is achieved by the use of settled effluent as a toilet flushing fluid. With the reduced volume of flow longer detention periods and higher treatment efficiencies are achieved. The amount of effluent is determined by the number and type of household plumbing fixtures connected to the system. Under certain conditions the unit can operate essentially as a closed system for long periods of time with little or no effluent.

In attempting to assess the present utility and potentialities of the individual household aerobic sewage treatment device, reference must be made to the fundamental objectives of safe disposal methods—the elimination of public health hazards and the prevention of nuisance. On the basis of present experience with these units it appears that with conservative design, normal loading, and adequate maintenance they may be operated without nuisance and will produce an effluent that is acceptable in appearance and free from objectionable odors. This is a notable achievement.

Existing experience, however, is not sufficient to establish the extent to which these treatment devices reduce the hazard of water-borne disease. Here important and complex problems arise that must be considered carefully in determining the proper methods for the disposal of the effluent from these new devices.

Some insight is provided by the efficiency of removal of coliform bacteria. There is a paucity of reliable information of this type, but even with more extensive data, coliform removal performance is by itself not sufficient to establish firmly the degree of health hazard associated with various schemes of disposal of the effluent; more information is needed. The coliform re-

moval efficiency of these devices relative to other benchmarks of coliform density is indicated in Table I.

State health departments and water pollution control organizations have generally stipulated that effluents from household, institution, and community sewage treatment plants discharged to natural drainage must have low coliform density levels approximating those required by bathing and recreation water codes. Occasionally installations such as sewage disposal systems in rural schools in sparsely settled regions are approved in which these levels are exceeded. Such approval, however, has been given only after a careful appraisal of local health hazards. Probably in no case, however, have health organizations knowingly permitted the discharge of sewage plant effluents with coliform densities in excess of about 10^5 per 100 ml to natural drainage, dry ditches, or small streams that are accessible to humans. If a plant effluent exceeds this approximate density level, then good practice requires conveyance of the effluent in a covered drain to a receiving water of capacity sufficient to provide adequate dilution and self-purification.

The most reliable data relating to coliform removal efficiency of individual household aerobic sewage treatment systems indicate that these units yield coliform densities in excess of those which have been considered tolerable for surface discharge in areas accessible to humans. If conventional sanitation practice is to be maintained, then further processing of the effluents of these units is needed. Aside from soil absorption methods that are considered in a subsequent section of this paper, possibilities for further treatment include (*a*) subsurface (or possibly in special cases superficial) coarse sand filtration, (*b*) aerobic stabilization in ponds or contact beds, and (*c*) disinfection or possibly pasteurization.

Sand filters offer a feasible solution in some cases but experience with these has not been entirely satisfactory. They require careful construction and may be expensive. Moreover, filters are often difficult to maintain and may require more or less periodic revamping or replacement. The quality of the effluent of sand filters varies widely in different installations depending on the depth, effective size, rate of filtration, and influent quality. In some jurisdictions chlorination of the effluent is required. Aerobic stabilization ponds, contact beds, and disinfection units at present are generally not suitable for use for individual household sewage but may be effective and practicable in small community treatment plants. Further research to develop practical household sewage disinfection systems is needed; such devices would fill an important gap in the present array of treatment methods and would undoubtedly have great utility in many intermediate and low population density areas. However, with any one of these alternatives the cost to the householder is increased substantially beyond that of the self-contained aerobic treatment unit. There is a line of research that would appear to be productive in partially overcoming this additional cost barrier. The authors believe that (*a*) with

TABLE I.—Coliform Density Comparisons in Various Media

Fluid	Approximate Range of Coliform Density per 100 ml
Household raw sewage	10^8 to 10^9
Septic tank effluents	3×10^7 to 3×10^8
Individual household aerobic units*	10^6 to 10^7
Septic tank with subsurface sand filters	3×10^4 to 3×10^5
Bathing water (typical standards)	10^2 to 10^3
Drinking water	0.1 to 1

* Based on limited data from systematic sampling and standard methods of coliform assay. Numerous data from grab samples reported in the literature exhibit a wide range of densities; in part the disparity of these results may be attributed to use of non-standard coliform sampling and analytical techniques.

careful design of baffling, compartmentation, and inlet and outlet arrangements, the degree of hydraulic short-circuiting, which is excessive in some commercial and experimental units, may be minimized so as to reduce substantially the hazard of passage of pathogenic microorganisms; and (*b*) with a source of heat and an efficient heat exchanger (thermophilic aerobic stabilization on the inflow side and minimal pasteurization temperatures on the outflow side) it is possible that a nuisance-free effluent could be produced that might safely be discharged to neighborhood surface drainage.

Trends in Septic Tank Design

No fundamental change in septic tank design has occurred during the past 30 years. However, there has been an increased recognition of the merits of certain design and maintenance criteria, and practice has become more nearly uniform throughout the country. There has been an increased use of steel tanks, although properly cured cast-in-place and precast reinforced concrete tanks are generally preferred. The recent adoption by manufacturers of steel tanks of minimum specifications embodied in Commercial Standard 177–51 of the United States Department of Commerce has resulted in improved construction.

The principal trend has been toward specification of larger tank capacities and the elimination in some regions of types of tanks having inferior hydraulic and structural characteristics. This improvement is reported extensively by Weibel (10). The salient results of this investigation included a determination of the following:

1. Optimum sizes (capacity per person or per bedroom).
2. Relative proportions of the total volume needed for sludge, scum, and intranatant liquor.
3. Baffling, inlet and outlet arrangements to eliminate excessive hydraulic short-circuiting and sludge erosion.
4. Additional capacity needed for households with food waste grinders and automatic sequence washing machines.

The results of this investigation and recent experience in various states are incorporated in a recent Public Health Service publication (11). All investigations have pointed up the need in septic tank operation for periodic inspection and cleaning at proper intervals to avoid excessive accumulation of sludge and scum. Otherwise solids will be carried out of the tank to the disposal field and will clog it. When the absorption capacity of the bed is exceeded it is necessary not only to clean the tank but also to renovate or even replace the absorption field.

Research in the past three years on high rate sludge digestion and stabilization at several universities and other experimental centers has indicated that the time required for anaerobic sludge digestion or stabilization may be substantially reduced by maintaining optimal conditions of temperature and pH, and providing proper control of mixing, recirculation, and hydraulic short-circuiting of the sludge.

Ettinger, Witherow, and Coulter (12) have reported the results of laboratory and field tests of anaerobic treatment of domestic sewage in upward flow tanks having detention periods of about 18 hr. The treatment efficiency of these devices is equivalent at least to that attained in conventional primary treatment and in some tests the effluent was comparable (on the basis of BOD and suspended solids concentration) to that of high rate trickling filters. The method appears to eliminate the need for separate sludge digestion and offers attractive possibilities for small interim community treatment plants.

While all these results have had application so far only to municipal and small community sewage treatment plant design, there is reason to believe that similar improvements may be achieved in household sewage treatment. These would undoubtedly involve radical reshaping of the anaerobic tank, perhaps with utilization of upward flow, mechanical mixing, and small amounts of heat.

Ludwig (13) has presented a penetrating analysis of field tests of tanks of various sizes and shapes that indicate possibilities for improved design. He has also suggested that enhanced performance might be attained by installation of a mechanical mixer in the primary chamber of a two-compartment tank which, operating periodically, would speed the process of anaerobic digestion.

Soil Absorption Systems

Since the effluent of septic tanks and other anaerobic and aerobic treatment devices for the wastewater of individual households generally cannot be "safely" discharged on the surface of the ground, and since further treatment before ultimate surface disposal is usually "expensive" and often "impractical," it is manifest that soil absorption systems will continue to be used extensively in areas where community sewerage and sewage treatment is "prohibitively" expensive. Soil absorption systems, however, are known to have serious limitations in areas having "poor" soils.

All the frequently used words in quotation marks in the previous paragraph in themselves have no semantic content in a quantitative sense except insofar as they can be defined and elucidated by an analysis of the costs and benefits (elimination of public health hazard and nuisance) of various sewage disposal schemes. Cost and benefit analyses of conventional sewerage and sewage treatment systems may be made with relative ease and reliability. Likewise cost estimates of septic tanks and household aerobic sewage treatment devices (including maintenance and operation costs) may readily and reliably be made. This is not true, however, for soil absorption systems. While there is much information relating to soil absorption systems in the literature and in the files of local and state regulatory organizations, the data have not been systematically analyzed. This is in part due to the circumstance that no satisfactory theory of the soil absorption system has been developed.

The objective of this section is to present a tentative theory of the functioning of soil absorption systems together with an economic "model" or formulation by which the merits and limitations of these systems may be measured. It is hoped that the theory will make it possible to close an important gap in engineering analysis and to delimit more precisely those zones in which community sewerage becomes "prohibitively" costly and to identify more definitively those zones where the soil is too "poor" for absorption systems. It must be emphasized, however, that in the sections to follow, it is methodology rather than practical results and conclusions that is being discussed. Reliable quantitative data are lacking and data used in the various examples are not necessarily firmly based on experiment and experience; they serve primarily to elucidate the theoretical argument.

Soil absorption systems include deep seepage pits, shallow seepage beds, narrow (and sometimes deep) seepage trenches, and conventional tile fields, which may have a single trench or several trenches in a variety of configurations. Combinations of these devices are used in some installations. All are *disposal* systems in that they produce no surface effluent. It is in this respect that they are to be distinguished from *treatment* devices such as sand filters and filter trenches

which do produce a surface effluent requiring ultimate disposal.

Although the terms "treatment" and "disposal" are sometimes loosely used synonymously, it is worthwhile here to assign different connotations to these terms. For example, septic tanks through anaerobic treatment reduce the clogging characteristics of raw sewage. Further treatment may occur depending on the length of detention in the storage zone of the absorption system prior to absorption. Disposal (here) is by percolation into the ground and by evapo-transpiration.

In some absorption systems, such as the cesspool, the processes of treatment (sludge deposition and anaerobic decomposition) and ultimate disposal by absorption are combined. In all soil absorption systems some retention or storage of the fluid occurs in the leaching device prior to absorption. Consequently, some additional anaerobic decomposition or even aerobic stabilization takes place. In certain devices, such as the tile field and the seepage pit, this additional treatment may be augmented by the presence of rocks or gravel which provide contact surfaces for biological activity.

Loading Capacity Factors

The loading capacity of an absorption system is determined by the following factors:

1. The effective area of interface between the soil and the liquid. Vertical surfaces are probably more effective than horizontal surfaces because (*a*) soil permeability in typically layered soils is greater in the horizontal plane than in the vertical direction, and (*b*) bottom areas are usually covered with a thicker sludge blanket than are side wall areas.

2. The permeability, porosity, and homogeneity of the soil. In soils of high permeability the movement by convection of the liquid away from the interface is relatively rapid. Soils of high porosity permit of a greater freedom of movement by diffusion of oxygen to the zone of biochemical stabilization; moreover, evaporation rates are higher in porous soils. Homogeneous soils have a greater capacity for absorption than non-homogeneous soils, other factors being the same, since these will allow shorter pathlines of movement of water by convection and air by diffusion. An impervious stratum near the ground surface may inhibit the downward diffusion of oxygen and the upward movement of water in evaporation and transpiration.

3. The geometry or shape of the system. The various types of soil absorption systems differ one from another chiefly by their shape. Tile fields and seepage beds are relatively shallow and have a low ratio of liquid volume to soil surface area. Seepage pits and cesspools are relatively deep and have a high ratio of liquid volume to soil surface area. This ratio provides a useful parameter for describing and classifying soil absorption systems as to size and shape. It is convenient to refer to the ratio as the "hydraulic radius" since there is a formal analogy with the hydraulic radius of open channels [(volume of water in a reach of channel)/(area of wetted contact interface)].

4. The hydraulic arrangement for loading different components of the absorption field. Sullivan, Coulter, and Bendixen (14) have presented the results of field studies showing that marked improvement in performance may be obtained in tile fields on sloping ground when the individual trenches are loaded in series over that obtained with the conventional arrangement of trench laterals dosed in parallel from a distribution box. In the latter system the useful life is determined by the failure time of that lateral having the smallest capacity; in the serial distribution system all trenches must be filled before failure can occur.

Ludwig (15) describes experience in California at installations where sewage is produced 24 hr/day (gasoline service stations, bus cafes, etc.). It has been found that where steady discharge to an absorption system causes clogging, that if the system is divided into two sections which are dosed alternately, no further clogging occurs. He suggests that the improved performance may be due to better ventilation.

As has been pointed out, soil absorption systems serve two functions: (*a*) they provide additional or supplemental liquid storage time for biochemical decomposition and stabilization which improves the filterability of the effluent, and (*b*) they provide contact interface for absorption. In shallow systems such as tile fields and seepage beds, which have a small hydraulic radius, function (*a*) may be relatively unimportant; in deep systems such as seepage pits, which have a large hydraulic radius, function (*a*) may be of considerable importance in determining the over-all efficiency.

Shape may affect the load capacity of an absorption system in another way that is not directly related to the hydraulic radius. An absorption system with a long "shore line" (i.e. line of contact between air, liquid, and soil) will be more efficient, other things such as soil area and hydraulic radius being equal, than a system with a short shore line, since the rate of diffusion of oxygen to the zone of biochemical stabilization near the liquid-soil interface will be greater in the former case. Another factor tending to increase the amount of biochemical stabilization in the soil along the shore line is the diurnal "tide" of the liquid surface, which speeds absorption and oxygen transfer and uptake in littoral zones. Also, there may be a substantial seasonal variation in the elevation of the liquid surface correlating with seasonal fluctuations the ground-water level that will enhance the rate of stabilization near the line of contact of the liquid, the soil, and the air. The conventional tile field with multiple trenches has a long shore line and this possibly may well be a reason that this device is often found (for a given type of soil) to have a higher capacity per square foot for receiving effluent than some other types of absorption systems.

Factors important in the selection of the proper type of absorption system include: (*a*) the average depth to the ground water level and seasonal fluctuations of this level, (*b*) the depth and thickness of previous strata, (*c*) lot size, (*d*) cost and availability of earth-moving machinery, and (*e*) accessibility for maintenance.

Fresh Water Percolation Tests

In current practice an approximate measure of the areal loading capacity of the soil for receiving septic tank effluent is obtained from falling-head percolation tests with fresh water. Several "recipes" for conducting this test and interpreting the results have been promulgated (10) (11) (16) (17) (18) (19). Empirical computational routines for reducing the raw percolation rates with data for unsteady falling-head conditions to quasi-stable "equilibrium" rates have been devised by a number of investigators to extract the maximum amount of information from short-time tests.

In certain soils the percolation test is known to have a fair correlation with long-term septic tank effluent absorption capacity and so provides a useful tool for design. In other soils the correlation is low or completely lacking despite the fact that absorption systems may work satisfactorily in these. Consequently it would appear that some important soil characteristic or characteristics is not taken into account by the usual type of fresh water percolation test.

There appears to be a widespread belief that the rate of absorption of

sewage is limited primarily by hydraulic friction in the soil and accordingly the percolation test should be designed to provide an estimate of this—especially to establish an "equilibrium" rate of fresh water seepage. On the basis of recent developments in the soil sciences there is some reason to question the assumption that hydraulic friction is the dominant and controlling factor in all soils. Investigations of the feasibility of ground water recharge with sewage and sewage plant effluents in California (20) have contributed greatly to knowledge of the process of sewage absorption in soils. The following interpretation of these important results is that of the authors and does not necessarily coincide with that of the investigators. It should be said, however, that the interpretation has only the status of an hypothesis and is useful only in designing experiments to establish facts.

In a new soil absorption system a film of sludge and biological gel is quickly built up on the liquid-soil interface, and organic and inorganic residues rapidly fill a portion of the interstices between the grains for a very short distance beyond the interface. It is in the zone of partial clogging that the hydraulic gradient is greatest. Only a small or negligible amount of hydraulic loss occurs in the soil regions beyond this "bottleneck," as may be inferred from the fact that the effluent percolation rates here are normally one or two orders of magnitude smaller than those of the fresh water percolation test. Moreover, in a new system the absorption proceeds at the greatest rate in the most pervious strata or sectors. The interfaces of these become clogged more quickly than the other areas so as to reduce the local percolation rate and to cause absorption to be more nearly uniformly distributed over all strata.

At the same time that particles of sewage solids are forming a clogged zone upon and a short distance beyond the interface, biochemical decomposition and liquefaction is proceeding. Biological activity is accelerated with the concentration of organic matter and creation of large areas of contact surfaces. The biochemical processes of liquefaction, gasification, and progressive stabilization of organic solids tend to restore the perviousness of the interfacial zone. The action may be greatly speeded according to the hypotheses under discussion if a sufficient amount of atmospheric oxygen is available so that a fairly complete degree of aerobic decomposition occurs. The principal mechanism for the movement of oxygen from the air to the submerged interface is diffusion through the soil. Computations indicate that even in relatively non-porous soils a steady rate of oxygen transfer sufficient to satisfy the BOD at the interface can be maintained with diffusion gradients obtained in many common types of soil absorption systems. Also, many absorption system designs provide adequate detention time prior to absorption for a high degree of aerobic stabilization to take place given an adequate supply of oxygen. Diffusion is, of course, greatest in porous soils; unlike convection, however, it does not necessarily take place chiefly through large diameter pores. The rate of movement of a gas by diffusion through a dry soil is independent of grain size and depends only on the concentration gradient of the gas and the void ratio of the soil. Convection rates of a fluid through a granular medium are markedly affected (for fixed porosity and pressure gradient) by grain size; the velocity of percolation varies approximately as the square of the grain size. If the diffusion of oxygen to the interface is in fact the rate controlling process in the absorption mechanism, it cannot be expected that the usual fresh water percolation rates will correlate well with measurements of the areal sewage load capacity of various soils. While fresh water

percolation rates do to some extent reflect soil porosity, they are markedly influenced by the presence or absence of pores of large diameter particularly if "equilibrium" rates are measured. If porosity (for oxygen diffusion) rather than permeability (for liquid convection) is the important soil characteristic, it would appear in the correlation analysis that initial fresh water percolation rates, which are influenced by capillary action and hence by small pores, should be used rather than the "equilibrium" rates which depend chiefly on the larger pores. In fine soils, particularly those with an admixture of clay, the diffusion of oxygen is of course inhibited by the fact that such soils retain moisture from rain which fills part of the interstitial space. Percolation tests do in part reflect the drainability of soils and this may well be the chief reason why some positive correlation is found between percolation tests and sewage load capacity of soils.

At present, knowledge is lacking as to whether effluent percolation friction in the soil or the transfer of oxygen to the soil-liquid interface is the rate limiting process. It may well be that both processes are important. Since, however, in many soils the results of fresh water percolation tests are known to yield unsatisfactory estimates of effluent areal loading capacity, it is evident that fundamental research is needed to elucidate the mechanism of soil absorption. If such research indicates that oxygen transfer is important then (*a*) soil absorption systems should be designed so as to obtain adequate ventilation at and near the liquid-soil interface; and (*b*) in testing soils to obtain an estimate of their areal load capacity for effluent, a technique should be used that measures soil porosity per se rather than permeability.

It is relevant that if oxygen diffusion in the soil were normally the rate limiting process, then the capacity of a given soil for receiving the effluent of a household aerobic sewage treatment device should normally be greater than its capacity for receiving septic tank effluent, because the former normally has a lower BOD and therefore a smaller amount of diffused oxygen will suffice to control the rate of clogging at the interfaces. This conclusion is corroborated by the results of ground water recharge experiments with settled sewage and with effluents from biological aerobic sewage treatment plants.

Mathematical Formulation of Areal Capacity of Soil Absorption Systems

The capacity of a soil for receiving wastewater is eventually exhausted because of the accumulation of inorganic and stable organic residues in the soil pores near the liquid-soil interface. The relation between the soil type, the organic loading rate, the area, and the useful life of the system is given by

$$L = fPt/A \quad \ldots\ldots\ldots (1)$$

where

fP = the "equivalent" population load. P is the number of persons in the household; and f is a factor reflecting (*a*) the type and extent of treatment of sewage, if any, prior to soil absorption; and (*b*) the presence or absence of food waste grinders or automatic washing machines in the household. Since the bulk of available experimental data pertains to septic tank effluents, it is convenient to set $f = 1$ for households with septic tanks of adequate capacity but without food waste grinders or automatic washers. If these latter are used then f will be larger than unity; the Manual of Septic Tank Practice (11) suggests appropriate numerical values based on present information. Also, if an

aerobic treatment unit is installed, f may be smaller than unity, possibly by a considerable amount. At present experimental data are not available to indicate the appropriate magnitude of f for these devices.

A = the effective area (in square feet) of the soil liquid interface in the absorption system when the ultimate permissible flow level (elevation) is attained in the absorption system.

t = the useful life (in years) of the system before the ultimate permissible flow level of the system is attained; continued operation will result in seepage to the ground surface or sluggish draining of household plumbing fixtures.

L = the unit areal load capacity of the soil. It reflects (*a*) the permeability and/or porosity of the soil; (*b*) the geometry of the system (hydraulic radius, and length of "shore" line); (*c*) the storage or detention time of effluent prior to absorption; and (*d*) the method of hydraulic control.* The parameter L varies in different soils for a given type of system, and it varies with different types of systems for a given kind of soil. Numerically, L represents the number of years that one square foot of soil-liquid interface can receive the wastewater of a single "equivalent" person before becoming permanently clogged. L may be measured directly from absorption system performance data by use of equation 1, or it may be estimated by a soil test, such as the fresh water percolation rate test.

The implications of Equation 1 may be elucidated by an example.

A six-person house has a septic tank (with adequate capacity and adequate maintenance) but no food waste grinder or automatic washing machine. A soil absorption system is installed consisting of a single seepage pit, 3 ft in diameter and 40 ft deep (sidewall area, 377 sq ft). The system is used 10 yr and then becomes clogged so that sewage appears on the surface of the ground. According to equation 1 if the household had consisted of 3 persons instead of 6, the system would have lasted 20 yr. Alternately, if there had been two seepage pits operated serially instead of one, the system would have lasted 20 yr with 6 persons. Also, if a food waste grinder (or an automatic washing machine) had been used, the system (with 6 persons) would have lasted less than 10 yr; if an aerobic sewage treatment device had been installed instead of a septic tank the system might have lasted longer than 10 yr.†

* The areal load capacity of a multiple unit system (e.g., having two or more trench laterals or seepage pits) depends in part on the method of hydraulic control of inflow. A multiple unit system will have a lower over-all L value if each unit as it is filled to maximum permissible flow level elevation is isolated hydraulically from the others, than if the hydraulic connection is retained while the other units are filling. The soil absorption capacity of a unit is not necessarily exhausted when it becomes filled for the first time. It may have a considerable amount of residual absorption capacity if inflow is applied at a reduced rate.

† An implicit assumption underlying Equation 1 is that the *organic* loading (pounds of BOD per day per square foot of soil contact area) is directly proportional to contributary population. This assumption is supported by a significant amount of data. These data indicate that the *hydraulic* loading (gallons per day per square foot of contact area) is *not* directly proportional to the population; e.g., the weekly volume of laundry wastewater from a family of six is generally less than twice the volume from a family of three. However, a multiple regression analysis of the available data indicates that the useful life of absorption fields is much more closely dependent on organic loading than on hydraulic loading. Therefore, Equation 1 has

For purposes of design it is convenient to write equation 1 in a different form

$$A_d = \frac{f_d P_d t_d}{L_d} \quad \ldots \ldots \ldots (2)$$

where the subscript d with f, P, t, and L indicates these are the design values predicted for these variates, and A_d is the predicted areal capacity of the absorption system to be used in the design.

The value of L_d in Equation 2 may be estimated from previous experience with soils in the community or it may be estimated from a soil test (or tests). The value of $f_d P_d$ is the predicated equivalent person loading as based on the number of bedrooms, the type of pretreatment, if any, before absorption, and the presence or absence of a food waste grinder and automatic washing machine. The quantity t_d is the desired (or desirable) life of the absorption system before it must be revamped or replaced. In the past in many states it has been customary to predicate a 20-yr life for the system and to design accordingly. An analysis of economic and public health aspects presented in the next sections indicates that this may be an uneconomic and possibly hazardous design predication in some situations. In this analysis it is assumed that no errors occur in the estimation of f, P, and L.*

Economic Analysis

The total cost of an individual household sewage disposal system can be divided into four component costs as follows:

1. Cost of installation of the septic tank or other type treatment unit.
2. Present value of future costs of maintenance, operation, and inspection of the treatment unit.
3. Cost of installation of soil absorption system.
4. Present value of future costs of renewal, i.e., periodic repair or replacement of the absorption system at regular intervals of t years.

In terms of present value the total cost, C_T, in dollars, is

$$C_T = C_1 + C_2 + C_3 + C_4 \ldots (3)$$

where:

C_1 = initial cost of installation of treatment unit, dollars.

$C_2 = c_2 \sum_{j=1}^{T} \frac{1}{(1+i)^j}$; where c_2 is the annual cost (in dollars) of operation, maintenance, and inspection of the treatment unit (e.g., for sludge removal or electric power); i is the discount or interest rate, and T the economic life (time horizon) of the system. The value of T used may depend in part upon whether and when it is expected that community sewerage and sewage treatment will be introduced.

C_3 = initial cost of installation of the soil absorption system, dollars.

C_4 = present value of future costs of periodic repair or replacement of the soil absorption system; renewal cost, dollars.

The initial cost of the absorption system, C_3, depends on a number of factors including the effective soil-liquid interfacial area needed. In most installations the cost per unit area decreases as the size of the absorption system increases. For example at a

been based primarily on organic loading. Some ancillary adjustment for atypical hydraulic loading may be made as needed by means of the parameter f in the formulation.

* The *actual* life of a soil absorption system designed in accordance with Equation 2 will differ to a greater or lesser extent from the expected (or design) life because of inevitable discrepancies between the actual values of f, P, and L, and those predicated in the design. The chief source of error lies in the estimation of L, the unit areal load capacity of the soil.

TABLE II.—Variation of Initial Cost with Design Area

Design Area (A_d) (sq ft)	Cost (C_3) ($)
0	33
100	61
200	89
300	117
400	145
500	172
600	200
800	256
1,000	311
1,200	367

particular site and using a given type of device such as a seepage pit, the variation of C_3 with area might follow the schedule shown in Table II.

Soil absorption systems costs vary over a wide range depending on the type of soil, the type of system, and the extent to which earth moving machinery is used. However, it often happens that the cost schedule as a function of absorption area can be approximated for practical purposes by a linear equation with terms for fixed and variable costs. That is,

$$C_3 = a + bA_d \quad \ldots\ldots (4)$$

where

a = fixed cost, dollars,

and

b = cost per unit of area of absorption surface, dollars per square foot.

If earth moving machinery for digging trenches and beds or for boring pits is used, the ratio of b/a will tend to be low. If the system is constructed manually, b/a will tend to be high, particularly if the cost of labor and material (e.g., tile pipe, gravel, or rock fill) is high. A wide range of values of a and b, and the ratio, b/a, occurs in different localities and with different types of soil absorption systems.

It is convenient to express the present value of future costs of periodic repair or replacement, C_4, in terms of the initial installation cost as

$$C_4 = (1 + \beta)C_3 \sum_{j=1}^{T/t} \frac{1}{(1 + i)^{tj}} \quad \ldots (5)$$

where

t = the useful life of the system in years, the length of the interval between successive replacements or repairs, or renewal period.

β = a factor that takes into account the possibility that repair costs may be less than initial costs ($-1 < \beta < 0$), or that replacement costs may be greater than the initial costs ($\beta > 0$). If replacement (or repair) costs equal the initial cost, then $\beta = 0$. The usual situation is that renewal costs exceed initial costs (disturbance of lawns and gardens, difficulty of access, etc.). However, if the reduction of the cost of renewal is explicitly made a design objective, it may be realistic to predicate β-values less than zero.

The total cost of the absorption field may be formulated from Equations 3, 4, and 5 as follows:

$$C_F = C_3 + C_4$$

or

$$C_F = a\left[1 + \frac{bf_d P_d t}{aL_d}\right]\left[\frac{K}{e^{kt} - 1} + 1\right] \quad \ldots (6)$$

where

e = 2.7183, the base of Napierian logarithms,

k = $\log_e (1 + i)$, the instantaneous discount (interest) rate, and

K = $(1 + \beta)\ (1 - e^{-kT})$.

In obtaining Equation 6 the series of terms appearing in equation 5 has been replaced by a single term representing their sum in accordance with the well-known formulation for the sum of a geometric series of finite length. By substituting increasing values of t of

1, 2, etc., in Equation 6 with given values of the other parameters, it will be found that C_F will be large for low values of t indicating the excessive cost of overfrequent renewal. With high values of t, C_F will become large as a result of a high initial cost, C_3, that is uneconomic in that it involves investment in soil absorption capacity that will not be used for many years. Between these two extremes there exists an optimum renewal time that minimizes the cost of the soil absorption system. If the design is based on this optimum renewal time, t_d, the maximum economic efficiency will be achieved. As will be discussed in the next section other factors in addition to economic considerations may affect the design. However, it is worthwhile now to examine the implications of the economic efficiency model of Equation 6 by means of numerical examples.

Example 1

Given the following data: $fp = 6$ equivalent persons, $i = 0.03$ (discount rate), $T = 25$ yr (economic life), $L = 0.16$ person-yr/sq ft (soil loading parameter), $a = \$33.33$ (fixed cost of soil absorption system), $b = \$0.278$/sq ft (unit cost of the soil absorption area), and $\beta = 1.0$ (renewal cost/initial cost = 2). Find the design period t_d, corresponding to the minimum cost. The computation indicates that the cost of the absorption system is minimized if a design renewal life of approximately 15 years is used (Table III). At this optimum life the over-all cost including future renewal costs is 2.87 times the initial installation cost. It may be noted that the total cost becomes excessively large if design periods of less than 5 years are used.

The assumption of a linear relation between the initial cost of installation and the absorption as in Equation 4 is not at all essential in the analysis. Any functional relation may be used. In fact, it is not necessary to formulate the installation cost, C_3, as a function of area. The optimum design period may be calculated by multiplying the initial cost of absorption systems of various sizes by the factor

TABLE III.—Cost Computations for Absorption System

t (yr)	C_3 ($)	C_F/C_3	C_F ($)
0	33.33	—	—
1	43.75	35.83	1,568
2	54.17	18.16	984
5	85.44	7.56	646
10	137.55	4.04	555
15	189.66	2.87	545
20	241.78	2.30	555
25	293.89	1.96	575
30	346.00	1.73	599
35	398.11	1.58	627
40	450.22	1.46	658

$$C_F/C_3 = \frac{K}{(1+i)^t - 1} + 1$$

The time period corresponding to the minimum product is the optimum design life. The linear relation of Equation 4 is useful in that it accords with cost data for many situations for which it is possible to separate fixed and variable costs. The ratio b/a varies widely depending on the type of system and extent to which earth-moving machinery is used. An examination of the first derivative of C_F with respect to time, dC_F/dt, obtained from Equation 6 indicates that when b (variable cost) is large and a (fixed cost) is small, t_d will be small. On the other hand if variable costs are low and fixed costs high, large values of t_d will be economical. The mathematical model also indicates that long design periods are optimal when f and P are small and L, β, and T are large. Longer design periods are usually economical when the discount rate is low. An examination of the second derivative of C_F with respect to time, d^2C_F/dt^2, obtained from Equation 6 indicates that if the quantity bfP/aL

is large the minimum cost is well defined particularly in designs having low values of t_d, i.e., there is a pronounced curvature of the cost vs. time function in the vicinity of the minimum cost point. On the other hand if the quantity bfP/aL is small, say less than 0.1, the minimum cost is poorly defined and all values of t larger than 5 or 10 years yield about the same value of C_F.

The determination of the optimum design period may be speeded by the use of a graph. In Equation 6 if the first derivative of C_F with respect to time is set equal to zero, the resulting equation may be written in the form,

$$kaL/bfP = (e^{kt} - 1)^2 e^{-kt} K^{-1} - (kt + e^{-kt} - 1) \ldots (7)$$

For a fixed value of kt, Equation 7 indicates a linear relationship between kaL/bfP and $1/K$. This relation is shown in Figures 1 and 2. For any given set values of the parameters a, b, L, f, P, β, i, and T, there exists a particular corresponding point on the graph; the coordinates of this point may be computed and the optimum value of kt_d read from the family of straight lines. Thus for the first example the abscissa of the point for

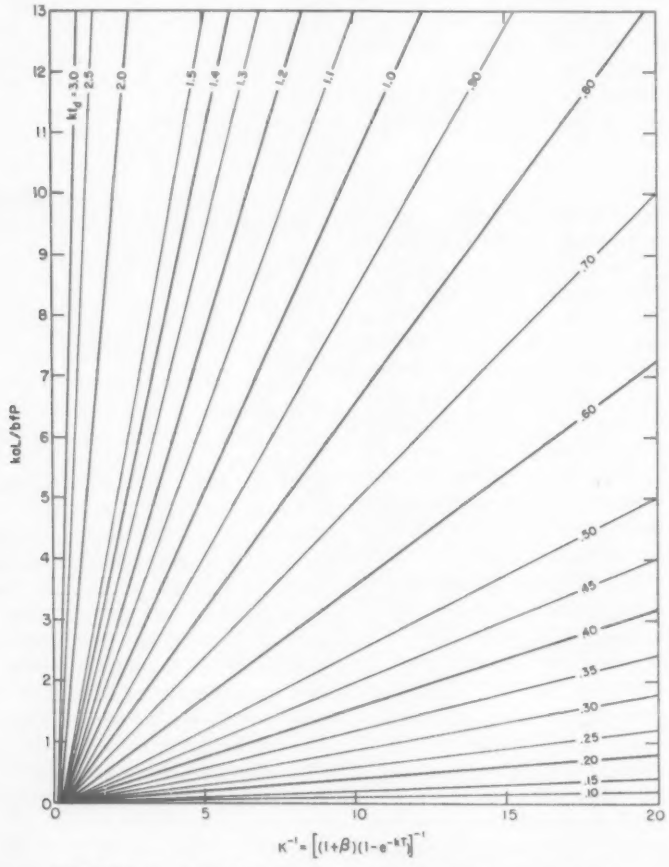

FIGURE 1.—Graphical representation of optimum design life.

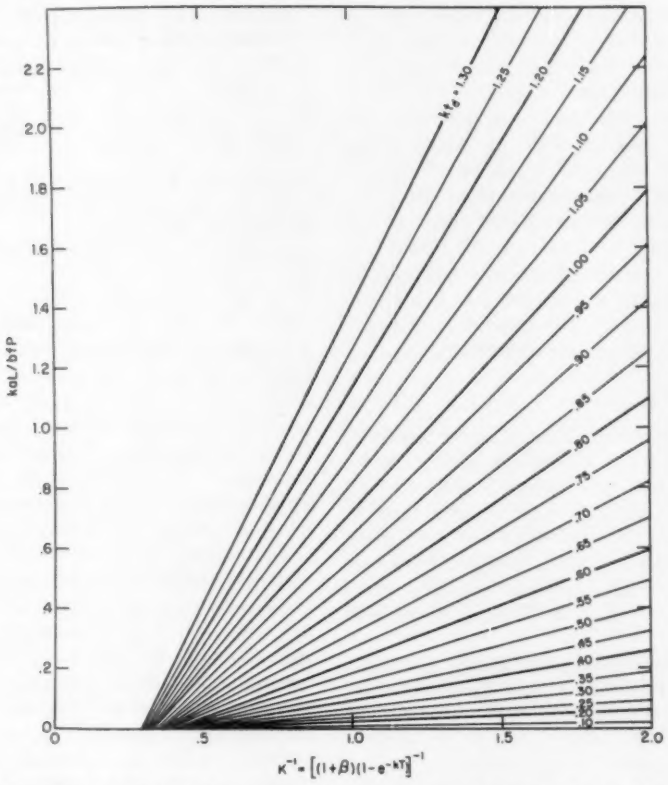

FIGURE 2.—Graphical representation of optimum design life for slightly different variable ranges than shown in Figure 1.

optimum design is $K^{-1} = 1.045^{-1} = 0.96$, and the ordinate is $kaL/bfP = (0.0296)\ (33.3)\ (0.16)/(0.278)(6) = 0.095$. From Figure 2 the value of kt_d is read as 0.42 and the optimum design life t_d is therefore 0.42/0.0296 or about 15 years.

Example 2

The effect of variation of the individual parameters of the mathematical model is indicated in Table IV which presents results of computations similar to that of Example 1 for 18 cases. A broad spectrum of optimum design periods occurs in the results. These range from 3.5 years in Case 12, which represents an installation in a poor soil, with a relatively low renewal cost and a high rate of discount, to a maximum of 40 years in Case 7, which represents an installation in good soil, with a relatively high renewal cost, and a low discount rate. It is manifest from these calculations that an "across the board" criterion of a fixed 20-yr design life for all systems as currently employed in some regions may be uneconomical and unjustifiable.

The examples of Table IV indicate the important effect of the renewal cost coefficient, β (as reflected in the parameter K), on the total cost and the renewal period. It would appear that serious consideration should be given in the original design of absorption systems (location, accessibility, and layout) to easing the work of repair

or revamping so as to reduce β and the total cost of the system. One technical possibility that would appear to merit investigation is the use of removable filter sections between the effluent and the soil and which would function to protect the latter against clogging. Conceivably these filter sections could be expendable and replaced from time to time; or they could be removed, cleaned, and restored to service. Either possibility could in principle eliminate the necessity of heavy and expensive reconstruction.

Example 3

The previous examples pertained to the design of absorption systems only. In these it was assumed that the cost of the treatment unit (septic tank or other device) was fixed and so did not affect the optimum design of absorption system. In this example three different types of sewage treatment-disposal systems are compared. System 1 includes a septic tank, system 2 an aerobic treatment unit, and system 3 a cesspool. In all three systems a seepage pit soil absorption system is used for ultimate disposal for which the following design data are assumed: a = \$33.33, b = \$0.278/sq ft, $L = 0.05$ (rather poor soil), $P = 6$ persons, $\beta = 1.0$, $i = 0.05$, and $T = 50$ years.

TABLE IV.—Summary of Results of Computations for Design Life for Eighteen Cases

Case	$\frac{bfP^{\alpha}}{aL}$	K	i	t_d (yr)	C_T† (\$)
1	0.1	0.5	0.03	15	158
2	0.3	0.5	0.03	7.5	326
3	1.0	0.5	0.03	5	826
4	0.1	1.0	0.03	22.5	222
5	0.3	1.0	0.03	15	512
6	1.0	1.0	0.03	7.5	1,422
7	0.1	2.0	0.03	40	314
8	0.3	2.0	0.03	27.5	800
9	1.0	2.0	0.03	20	2,432
10	0.1	0.5	0.06	10	108
11	0.3	0.5	0.06	5	206
12	1.0	0.5	0.06	3.5	482
13	0.1	1.0	0.06	15	142
14	0.3	1.0	0.06	10	302
15	1.0	1.0	0.06	5	790
16	0.1	2.0	0.06	25	188
17	0.3	2.0	0.06	17.5	444
18	1.0	2.0	0.06	12.5	1,290

* 0.1 = good, 0.3 = avg, and 1.0 = poor.

† Assuming as in Example 1, a = \$33.33 and b = \$0.278/sq ft.

TABLE V.—Cost Assumptions for Total Cost Analysis of Three Systems Using Poor Soil

System	Installed Cost (\$)	Operation Cost (\$)
1	350	201*
2	700	640†
3	4,022‡	—

* Cleaned at 3-yr intervals, \$35 per cleaning.

† \$25/yr for electric power plus \$10/yr for maintenance and inspection.

‡ Total cost obtained from equation 6.

Equivalent load factors for effluent of treatment units are assumed * to be, $f = 1.0$ for the septic tank, System 1; $f = 0.333$ for the aerobic treatment device, System 2; and $f = 3.0$ for the cesspool, System 3. Analyses of the present value of costs of the three types of treatment-disposal systems in accordance with the equations 3 and 6 reveal that (*a*) for system 1 (septic tank plus seepage pit) the total cost is \$1,981, (*b*) for system 2 (aerobic treatment device plus seepage pit) the total cost is \$1,876, and (*c*) for system 3 (cesspool) the total cost is \$4,022. These costs include the assumptions given in Table V.

These results together with similar computations for soils $L = 0.16$ (average soil) and $L = 0.48$ (good soil) are presented in Table VI.

With the hypothetical cost and load-

* The f-values used for the cesspool and the aerobic treatment device in this hypothetical example are based on a few scattered field observations and laboratory tests. However, these data are altogether insufficient to establish average or typical values for the loading parameter f. Therefore, this example merely exhibits the method of application of the formulation; it does not yield valid cost comparisons based on firmly established experimental data.

TABLE VI.—Hypothetical Cost Comparisons of Three Different Systems of Treatment-Disposal Installations in Three Types of Soil

System	Poor Soil $L=0.05$			Average Soil $L=0.16$			Good Soil $L=0.48$		
	l_d (yr)	C_F ($)	C_T ($)	l_d (yr)	C_F ($)	C_T ($)	l_d (yr)	C_F ($)	C_T ($)
1 (Septic tank + seepage pit)	12.5	1,430	1,981	17.5	508	1,059	27.5	212	762
2 (Aerobic treatment device + seepage pit)	17.5	536	1,876	27.5	212	1,351	37.5	104	1,192
3 (Cesspool)	7.5	4,022	4,022	12.5	1,348	1,348	17.5	508	508

ing data assumed in the examples shown in Table VI the septic tank system is the least expensive installation for the average soil. In this soil a system using a suitable aerobic treatment device would be competitive with the septic tank system if the cost of the treatment device could be reduced by $300.00. In the good soil range the cesspool is considerably more economical than the other systems, but in poor soil its cost becomes excessive. In the poor soil range the aerobic treatment device appears to offer economies; these, however, must be regarded only as possibilities pending experimental determination of the filterability of the aerobic effluent. It is pertinent to note that the design life of the absorption system increases as the soil condition improves and as the organic load —in this example as reflected by the factor f—decreases.

Example 3 indicates the method by which the mathematical model may be used as a guide to selection of the proper type of system. The particular rankings of types of systems based on cost as obtained in this example may be altered when different sets of design parameters are used. Since the aerobic treatment systems may prove to be less expensive than the conventional septic tank system under certain conditions, research is needed to establish appropriate values of the loading coefficient f for these systems.

It is believed that the current proscription of the cesspool in some jurisdictions should be reexamined. This device in principle can function satisfactorily in a fairly wide range of soil types if adequate depth is available. The prejudice against the cesspool derives in part from a history of unsatisfactory results experienced in the past with improperly designed installations in unsuitable soils and subsoil formations. Some regulatory codes reflect apprehension regarding the pollution and contamination of ground water by cesspools. Recent investigations of bacteria die-away and self-purification in ground water flow have shown such apprehension to be unwarranted under many conditions. There are, to be sure, certain formations (crevieed rock, etc.) where a cesspool installation could be dangerous.

It would appear that modern experience with the construction and operation of seepage pits and seepage trenches and a better understanding of the mechanism of soil absorption and of the economics of renewal could be applied to the cesspool so as to reinstate it as a respectable treatment device useful in defined situations.

Administration and Public Health Aspects

From the standpoint of a public health regulatory organization the economic analysis of the preceding section, which is concerned with the cost to the builder and ultimately to the householder, must be elaborated to

take into consideration the costs of administration, inspection, and enforcement pertaining to individual household sewage disposal systems.

It is worthwhile to direct the attention of American sanitary engineers to practice in other nations such as Switzerland and Sweden where administrative patterns have developed which place heavy reliance on the efficacy of systematic, detailed, and periodic inspections of household disposal systems, together with long-accepted, efficient, and orderly legal processes for enforcement. Inexpensive tank cleaning and system maintenance are provided by service organizations that are a part of (or licensed by) the local departments of public works. These practices, of course, entail greater governmental expenditure but in many communities they have been found to be more economical than the extension of the water carriage system to areas of low population density.

From the health viewpoint it is recognized that during the interval between the time of failure and time of repair of a system there is apt to be a health hazard to the community. In the long-run the level of hazard from malfunctioning systems depends on (a) the renewal periods of the systems, (b) the length of the time between system failure and repair, (c) the population density, and (d) the quantity, quality, and ultimate fate of effluent discharged on the surface.

The decisive consideration in the control of administrative costs and health hazards resides in policy relating to renewal period. While data needed to make a complete quantitative analysis of costs and benefits are lacking, a limited extension of the theoretical model is helpful in elucidating the problems of health departments in setting and administering regulatory codes pertaining to household sewage disposal. This extension is also useful in carrying out detailed cost analyses to establish the boundary zone between the utility of community sewerage and that of individual household sewage disposal systems.

The budgetary cost of administration, inspection, and regulation enforcement of individual systems which is ultimately reflected in tax rates, may be conceived of as a more or less fixed cost plus a variable cost that depends on the frequency of inspection and of failure. In mathematical language the present value of governmental cost may

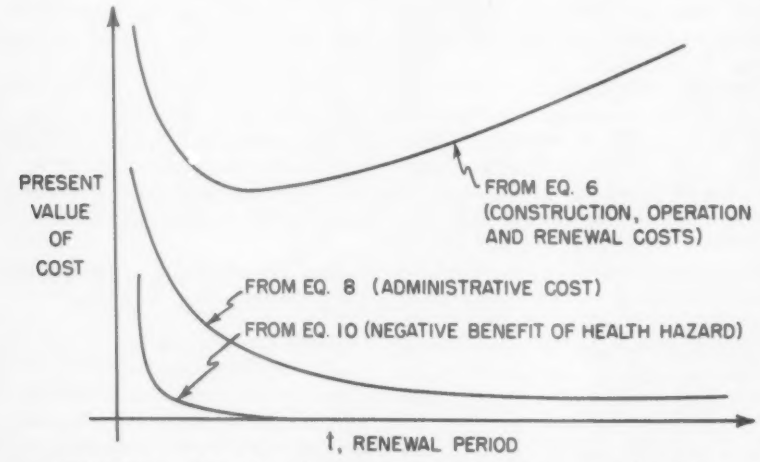
PRESENT VALUE OF COST
FROM EQ. 6 (CONSTRUCTION, OPERATION AND RENEWAL COSTS)
FROM EQ. 8 (ADMINISTRATIVE COST)
FROM EQ. 10 (NEGATIVE BENEFIT OF HEALTH HAZARD)
t, RENEWAL PERIOD

FIGURE 3.—Administrative cost as a function of the renewal period.

be formulated approximately as

$$nC_G = nC_5 + c_5 \sum_{k=1}^{n} \sum_{j=1}^{T/j} \frac{1}{(1+i)^{t_k j}} \quad (8)$$

where the first term on the right represents the fixed cost of administration of n installations in the jurisdiction including initial inspection, and the second term represents costs of periodic inspection which depend on the duration of the renewal periods; c_5 is the unit average cost of inspection of individual installations and it may include the cost of soil testing. The values of the discount rate i and the economic time horizon T used in governmental cost analyses are not necessarily the same as those predicated in the cost analysis for construction and operation (Equation 3).

The administrative cost of an individual system with a renewal period of t years may be expressed as

$$C_G = C_5 + c_5 K'/(e^{kt} - 1) \quad (9)$$

where

$$K' = 1 - e^{-kT}$$

This cost is shown diagrammatically in Figure 3 as a function of the renewal period.

The remaining element to be considered in the over-all benefit-cost analyses of individual household disposal systems is the residual public health hazard that exists even with a set of well designed and properly administered installations. Such residual health hazard also obtains in communities with sewerage and sewage treatment as a result of failures, accidents, and human errors, and also of small but finite risks inherent in the design of all sanitary engineering works. The efficiency of treatment devices can be made only to approach but not to attain perfection. In both the water carriage and individual household disposal systems these residual health hazards may be made negligible at a price. In the case of the individual system, the residual health risk is closely associated with the length of the renewal period incorporated in the design. Health hazards are approximately proportional to the frequency and duration of failures. By designing for long renewal periods and by maintaining an adequate program of administrative control, the frequency and duration of failures can be made low. However, this is done at the expense of increased investment and tax rates. In principle the residual health hazard may be included in the cost analysis by adding a term to the right hand side of Equation 6 of the form

$$C_H = c_6 \sum_{j=1}^{T/j} \frac{1}{(1+i)^{tj}} = c_6 K'/(e^{kt} - 1) \quad (10)$$

where c_6 is the average damage per failure (in dollars). This term, which decreases with increasing renewal periods, is shown in Figure 3. Alternately, it could be regarded as a negative benefit associated with the failure frequency corresponding to the stated design renewal period.

When the costs of administration and residual health hazard are combined with the costs of construction and operation, the over-all cost of the individual household disposal system from Equations 3, 6, 8, and 10 may be formulated as

$$C_T = C_1 + C_2 + C_5 + \left(a + \frac{bfPt}{L}\right) \times \left(\frac{K}{e^{kt}-1} + 1\right) + \frac{(c_5 + c_6)K'}{e^{kt}-1} \quad (11)$$

The inclusion of administrative and residual health hazard cost places a premium on longer design renewal periods. The effect is the same as increasing the value of the parameter a in Equation 6 and in the graphical method of determining the optimum design period (Figures 1 and 2). In

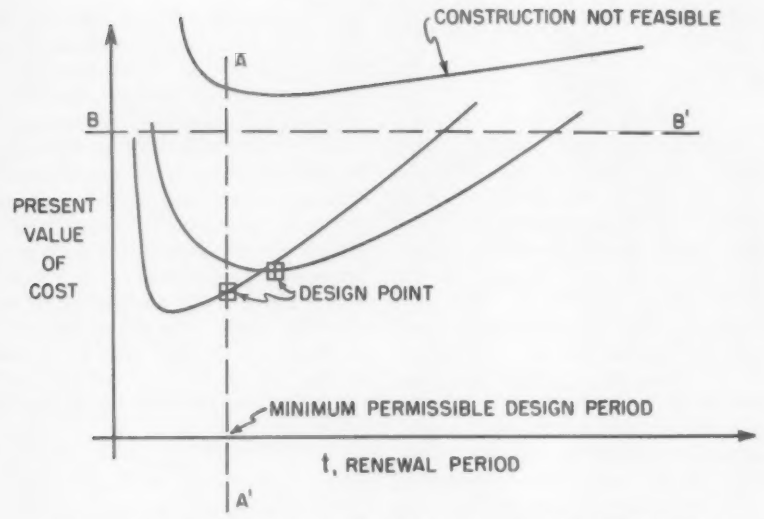

FIGURE 4.—Design curves showing minimum permissible design period.

communities where an estimate of the average administrative cost per inspection may be made, the term a in equation 6 should be replaced by the combined term, $a + c_5$. With this substitution the design will be adjusted approximately to take into account governmental costs. Since at present there is generally not a sufficient amount of information available to establish c_5 and c_6 quantitatively with a satisfactory degree of precision, a practical method for taking into account administrative and health costs and controlling them within reasonable limits would appear to be the establishment by health regulatory organizations of a minimum permissible design renewal period. Such a limiting value is indicated schematically in Figure 4 by the intersection of the vertical dashed line AA' and the horizontal time axis. With this constraint an optimal renewal period minimizing the over-all cost of construction, operation, and renewal (Equation 6) would be used in actual design if and only if it exceeded the minimum permissible value. Otherwise the latter would be used in the design.

Another constraint upon the use in design of the t-value minimizing C_F in Equation 6 arises from the practical consideration that the theoretical optimum renewal period may entail an initial cost that exceeds the financial resources of the builder or a total cost that is excessive for the owner. This constraint is shown diagrammatically on Figure 4 by the horizontal dashed line BB'. If the cost curve falls entirely above this constraint line, as it may with poor soils or otherwise unfavorable factors, then the site would not be used.

With these constraints and adjustments the economic model of Equation 6 may be employed for system design. Optimal values of t_d obtained by the graphical method would be employed if the coordinates, t_d and C_F, fall in the quadrant of Figure 4 to the right of line AA' and below the line BB'. Otherwise the minimum permissible design period as established by the health organizations having jurisdiction would be used in setting the design and in investigating the feasibility of construction.

Recommended Research

Applicability of Present Microbiological Quality Standards

In planning a research program to advance the technology of individual household sewage disposal systems the question arises as to whether the design objectives in use at the present time for municipal sewage disposal retain validity for contemporary conditions of life in metropolitan fringe areas. These design objectives stipulate (*a*) that the disposal method be such as to remove completely the sewage or effluent from the human environment (e.g., absorption into the subsoil); or that (*b*) the treatment method be sufficiently thorough to maintain all receiving waters accessible to humans at a quality comparable to that stipulated in regulatory codes for bathing and recreation waters. Professional opinion is not unanimous as to the applicability of these design objectives in areas of relatively low population density, and it is pertinent to discuss briefly some of the issues.

Good municipal drainage has been a prerequisite for healthy urban life since the time of the early city-states of Mesopotamia. One of the principal achievements of the sanitary revolution of the nineteenth century was the renascence and elaboration of the water-carriage system for the disposal of municipal sewage. The main channels of sanitary engineering endeavor in the present century have been directed toward improving and extending its utility. As a result of this effort, a remarkable reduction has occurred in the incidence of water-borne disease. Sanitary engineers are justly proud of this achievement. Concomitantly there is a propensity to regard community sewerage as being an almost indispensable feature of satisfactory liquid waste disposal schemes. Many engineers have serious doubts as to the validity of any trend to depart from the community water-carriage system. Historically, there have been many instances of failure of alternate schemes that have resulted from technical flaws and administrative difficulties. There is general professional agreement that the best direction for development of all urban and most suburban areas lies in the extension of the water-carriage system.

Some engineers and administrators, while not questioning the utility of community sewerage in areas of high population density, believe that there are and will be many exurban zones and diffuse fringe areas where the population density will be low for many years, and that rather than to insist on sewerage for these areas it is incumbent on the sanitary engineering profession to develop for them an improved technology to meet the requirements of safe sanitation practice. The principal issue here is whether or not the criteria for safe practice that have been evolved for sewerage systems in urban areas apply with equal force to areas with somewhat different environmental conditions.

Questions have been raised as to the degree of on-lot treatment needed and as to the relative safety of various schemes for ultimate disposal. It has been pointed out that most of the currently used microbiological standards of effluent and stream water quality were formulated in the first two or three decades of the present century when mortality and morbidity rates from water-borne disease were much higher and the arts of water treatment and disinfection were less advanced. It is true that quality standards such as those pertaining to coliform density, which were formulated then and have persisted with little change to the present time, were not in fact determined by calculations involving hazards of disease transmission, but were rather fixed on an "after the fact" basis following the demonstration of what early

water and sewage plants could achieve both in terms of effluent quality and reduction of disease. Furthermore, it has been noted that sewage now is less infective in some respects and that modern medicine with its potent antibiotics provides insurance against the outbreak of explosive epidemics of several diseases that formerly occurred. Some have argued from these premises that current quality standards can safely be relaxed for application in fringe areas.

On the other hand, many public health workers are concerned with the increasing incidence of hepatitis and the possible danger of other enteric virus infections. They believe that further epidemiological research may well indicate that effluent and stream quality standards should be made more stringent in the future. Moreover, it should be pointed out that although the infectivity of domestic sewage and treatment plant effluents has decreased with the reduction of certain water-borne diseases, there has been a concomitant reduction in the natural immunity of the population for these diseases, so that if sanitation safeguards were relaxed the hazard could be even greater than it was formerly.

An undisputed fundamental design objective in all treatment and disposal systems is continuity of operation and efficiency. In this connection there is some divergence of engineering opinion in the development of individual household sewage treatment systems as to how much reliance may be placed on mechanized components of treatment devices. It is true that radical changes have occurred as the result of mechanization in transportation, food-processing, and construction. There has been a universal acceptance of mechanically successful household devices such as the automatic washer, the refrigerator, the food waste grinder, and the power lawn mower. On the other hand, in many communities there has been a poor record of maintenance of household sewage disposal devices not having mechanical components. Proper long-term maintenance would become even more essential if mechanized devices were generally adopted. In considering the validity of the analogy to other technical arts it must be emphasized that mechanization in these has been predicated on the simultaneous development of efficient, reliable, and economical repair and maintenance services. Since the repair and maintenance of pumps, compressors, and motors transcends the ability of the average householder, effective servicing is a *sine qua non* for the admission of mechanized on-lot treatment devices.

In consideration of the hygienic and economic significance of safe sewage disposal in fringe area communities, it is of national importance that an intensive effort be made to obtain engineering and epidemiological data pertaining to health hazards associated with various systems of household and community sewage disposal.

Investigations of the degree of persistence of water-borne pathogenic microbes in passage through treatment plants, and their survival in waterways and on land surfaces are difficult, expensive, and often inconclusive. Carefully programmed research demands the utmost skill in analytical techniques and intelligence in statistical interpretation. Field studies, pilot, and laboratory tests of new or modified treatment devices and disposal systems require intricate experimental design and meticulous execution over long periods of time. However, it is only through such directed research that rational and definitive quality standards and design objectives can be set.

The principal conclusions of this report are: (*a*) that an intensive research and development program to advance the technology of sewage disposal for metropolitan fringe areas is needed; and (*b*) until such time as

the results of this program have become firmly established, conservative, time-proven sanitation objectives should be rigorously maintained in these areas.

List of Research Projects

1. Continued development of new and modified on-lot treatment methods.

(a) Optimal size, shape, and compartmentation for aerobic deposition-digestion units and anaerobic contact units. Possible use of an upward-flow arrangement. Determination of the utility of (*a*) heating and (*b*) mechanical mixing in the sludge storage zone.

(b) Optimal size, shape, baffling, and compartmentation for aerobic stabilization-sludge return units. A strong effort should be made to develop designs that minimize hydraulic short-circuiting.

(c) Detailed information relating to the physical, chemical, and microbiological characteristics of the effluent of aerobic and anaerobic treatment devices operated under a wide range of loading, operating, and climatic conditions.

(d) Reliable data pertaining to long-term operation and maintenance costs of aerobic and anaerobic treatment systems. Determination of the proper operating cycle for the aeration mechanism for various conditions of loading.

(e) New or modified construction materials for tanks and appurtenances for greater economy and longer life.

(f) Designs for tanks and appurtenances to facilitate inspection, cleaning, and repair.

2. Development of a practicable individual household sewage disinfection (oxidizing chemicals automatically dosed or possibly generated by electrolytic decomposition of sewage and brine; pasteurization is a possibility, perhaps coupled with heat treatment to speed stabilization).

3. Quantitative assessment of factors governing the self-purification of polluted ground water and the persistence of ground-water contamination. Particular attention to the fate of synthetic detergents discharged into ground-water strata.

4. Evaluation of the amount of evaporation and transpiration occurring in various types of soil absorption systems under different climatic conditions. Investigation of methods and designs to speed evaporation.

5. Fundamental investigation of the soil clogging process in absorption fields.

(a) Comparison of clogging rates with various types of effluents (aerobic, anaerobic, filtered, unfiltered, etc.).

(b) Effect of geometry of absorption system (e.g., pits vs. trenches) on clogging rate. Relative uptake rates on horizontal vs. vertical soil surfaces.

(c) Effect of soil characteristics (particle size distribution, porosity, permeability, etc.) on clogging rate.

(d) Influence of ventilation on clogging process.

(e) Influence on clogging rate of additional stabilization of effluents in storage zone of leaching devices (effect of gravel fill and other contact media on stabilization rate).

(f) Effect of diurnal alternate wetting and drying of soil-liquid interface on effluent uptake capacity of soil for effluents.

(g) Effect of temperature, pH, and sunlight on soil uptake rates.

6. Practical design of absorption fields (based on results of Item 5).

(a) Optimal shape of absorption fields (hydraulic radius, length of shore-line, etc., as discussed earlier in the report) as determined by existing soil and subsoil conditions in the field.

(b) Relative merits of serial vs. parallel dosing of leaching devices with more than one trench or pit.

(c) Advantage, if any, pertaining to intermittent dosing with weekly, monthly, yearly, etc., cycles.

(d) Utility of gravel packing and lining in leaching devices.

7. Practical methods for predicting soil absorption capacity for use in design.

8. Practical techniques for restoring permeability of absorption systems.

(a) Cleaning of interface and clogged zone of soil (mechanical, chemical).

(b) Expendable filter sections in pits and trenches to protect interface. Removable sections that can be cleaned and replaced.

9. Systematic analysis of costs of effluent leaching devices.

(a) New construction

(1) Fixed costs, (2) size-dependent costs.

(b) Revamping

(1) Fixed costs, (2) size-dependent costs.

10. Determination of costs of inspection and soil testing.

11. Delineation of utility of sand filters and stabilization ponds (covered and uncovered) to provide final treatment prior to discharge of effluent on ground surface.

Acknowledgments

In surveying a wide area of professional concern and in looking for future trends, it is manifest that the authors have relied on the cooperation of many individuals in local, state, and Federal agencies, in industry, and in private practice. It is not possible to acknowledge individually all who have expressed views and contributed information. To these persons an omnibus acknowledgment is made. Certain individuals, however, have been particularly helpful. The authors wish to acknowledge with appreciation the contributions and cooperation of Professor Percy H. McGauhey of the University of California, Mr. Clarence S. Slater of the Department of Agriculture, Mr. James R. Simpson, and Mr. Bernard T. Craun of the Federal Housing Administration, Mr. Bernard B. Berger and Mr. Malcolm C. Hope of the Public Health Service, and Mr. Harvey F. Ludwig, consulting engineer.

Many of the background technical data from which the theoretical framework has been constructed have derived from projects at the Robert A. Taft Sanitary Engineering Center. These projects have been supported by the Federal Housing Administration and full acknowledgment is made of the contribution of that agency in advancing the frontiers of knowledge and practice in fringe area sanitation.

In this paper views, inferences, and interpretations have been presented that do not necessarily coincide with those of the forenamed individuals and organizations; the authors hasten to excuse them from complicity in ideas that may be diametrically opposed to their own.

Finally, the authors wish to acknowledge the contributions of members of the Subcommittee on Waste Disposal and the Committee on Sanitary Engineering and Environment of the National Research Council, whose opinions and experience have done much to shape the views and conclusions of this report.

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SEWER MAINTENANCE MANUAL DUE SOON

Federation Manual of Practice No. 7, "Sewer Maintenance," the result of work by the Subcommittee on Sewer Maintenance with T. W. Clapham as Chairman, is now in the hands of the printer. Final publication is expected early in 1960. Watch the Journal for further announcements.

This manual will contain approximately 70 pages of well-illustrated and documented information on sewer maintenance.

Orders are now being accepted in the Federation office at the members' price of $1.00, or non-members' at $1.50. See page 76a for details of ordering. Orders on hand will be filled promptly when the publication is received.

BOD VELOCITY CONSTANT OF BOSTON-SOUTH METROPOLITAN SEWAGE

By Andrew P. Fisichelli and Vincent A. Palombo

In Charge of Laboratory and Senior Chemist, Nut Island Sewage Treatment Plant, Metropolitan District Commission, Quincy, Mass.

The value of the velocity or reaction-rate constant K* used in the first-order formula for the BOD reaction generally has been questioned in the literature. The original work of Phelps (1) in connection with relative stability resulted in the establishment of a K value of 0.1 for Boston and Washington sewages which has withstood the test of the years in value, use, and understanding. In recent years, however, it has been suggested that Phelps' value of 0.1 should be revised to be in keeping with present-day sewages.

Williams (2) in 1951, during his investigation of Minneapolis-St. Paul sewage, outlined a method for the establishment of K values by plotting (a) the ratio of the percentage of the ultimate BOD satisfied in 3 and 5 days against (b) the K value substituted in the BOD equation used to compute these very percentage values. The plot forms a curve from which K values can be read by simply entering the graph with the ratio of the actual 3-day to 5-day BOD. In brief, the work of Williams was repeated for the determination of present-day K values on typical Boston-South Metropolitan sewage entering the Nut Island sewage treatment plant. The flow is from a combined system serving 18 cities and towns including many small diversified industries and is at an annual average rate of 125 mgd, with seasonal flows ranging from 100 to 200 mgd. The flow undergoes primary treatment before discharge to the ocean.

Procedure

Establishment of Curve

The data used in constructing the velocity-constant graph (Figure 1) are listed in Table I. They were computed by substituting different values of K

TABLE I.—Data Used in Establishing Figure 1

K Value	Ratio of 3-Day to 5-Day BOD
0.050	0.669
0.075	0.699
0.100	0.730
0.125	0.758
0.150	0.785
0.175	0.810
0.200	0.832
0.225	0.853
0.250	0.871
0.275	0.887
0.300	0.903
0.325	0.916
0.350	0.928

* To the base 10.

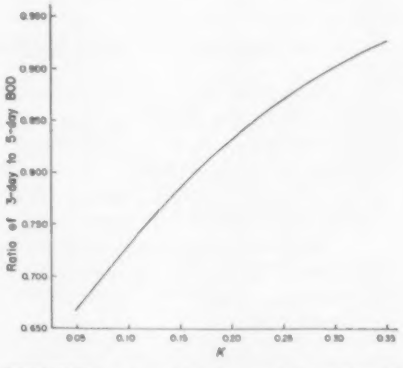

FIGURE 1.—K values for various ratios of the 3-day to 5-day BOD.

in the relationship:

$$\frac{y_3}{y_5} = \frac{1 - 10^{-3K}}{1 - 10^{-5K}} \quad \ldots\ldots\ldots (1)$$

where y_3 and y_5 are the 3-day and 5-day BOD, respectively.

Observed K Values

A total of 170 24-hr composite samples (85 raw sewages and 85 primary effluents) were collected and analyzed for their 3- and 5-day BOD according to "Standard Methods" (3). Collecting and testing took about one year. Sampling, however, was confined to those periods during which prechlorination was not employed. The K values were read from Figure 1 after the ratios of the 3- to 5-day BOD were computed.

Table II shows the grouping of observed K values for all raw sewage samples in order of magnitude and frequency. These values ranged from a low of 0.057 to a high of 0.285. Figure 2 is a probability plot of the data from Table II. The mean K

TABLE II.—Grouping of All Raw Sewage Samples

K Value	No. of Observations	Cumulative No. of Observations	Per Cent of $n + 1$ Observations
0.05 (0.040–0.059)	5	5	5.8
0.07 (0.060–0.079)	5	10	11.6
0.09 (0.080–0.099)	12	22	25.6
0.11 (0.100–0.119)	11	33	38.4
0.13 (0.120–0.139)	6	39	45.2
0.15 (0.140–0.159)	10	49	57.0
0.17 (0.160–0.179)	10	59	68.6
0.19 (0.180–0.199)	9	68	79.1
0.21 (0.200–0.219)	8	76	88.4
0.23 (0.220–0.239)	3	79	91.9
0.25 (0.240–0.259)	3	82	95.3
0.27 (0.260–0.279)	2	84	97.8
0.29 (0.280–0.299)	1	85	98.8

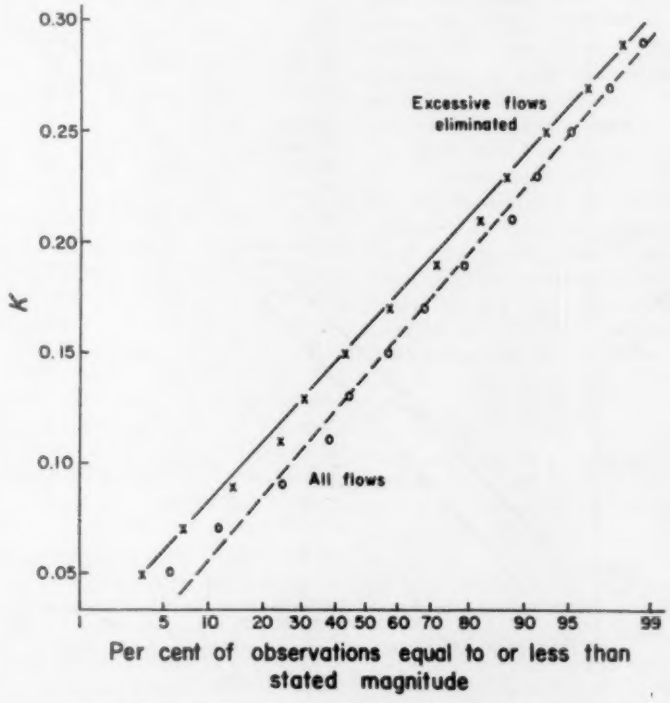

FIGURE 2.—Probability plot of K values for raw sewage.

TABLE III.—Grouping of Normal-Flow Raw Sewage Samples

K Value	No. of Observations	Cumulative No. of Observations	Per Cent of $n + 1$ Observations
0.05 (0.040–0.059)	2	2	3.5
0.07 (0.060–0.079)	2	4	7.0
0.09 (0.080–0.099)	4	8	14.0
0.11 (0.100–0.119)	6	14	24.6
0.13 (0.120–0.139)	4	18	31.6
0.15 (0.140–0.159)	7	25	43.9
0.17 (0.160–0.179)	8	33	57.9
0.19 (0.180–0.199)	7	40	70.2
0.21 (0.200–0.219)	7	47	82.5
0.23 (0.220–0.239)	3	50	87.7
0.25 (0.240–0.259)	3	53	93.0
0.27 (0.260–0.279)	2	55	96.6
0.29 (0.280–0.299)	1	56	98.2

TABLE IV.—Grouping of All Primary Effluent Samples

K Value	No. of Observations	Cumulative No. of Observations	Per Cent of $n + 1$ Observations
0.05 (0.040–0.059)	1	1	1.2
0.07 (0.060–0.079)	8	9	10.5
0.09 (0.080–0.099)	6	15	17.4
0.11 (0.100–0.119)	7	22	25.6
0.13 (0.120–0.139)	9	31	36.0
0.15 (0.140–0.159)	12	43	50.0
0.17 (0.160–0.179)	12	55	64.0
0.19 (0.180–0.199)	9	64	74.4
0.21 (0.200–0.219)	7	71	82.6
0.23 (0.220–0.239)	4	75	87.2
0.25 (0.240–0.259)	4	79	91.9
0.27 (0.260–0.279)	4	83	96.5
0.29 (0.280–0.299)	2	85	98.8

value is seen to be 0.140, and the standard deviation is ± 0.066. Since the period of testing necessarily included days of exceptional storm water flows, it was thought desirable to group K values for samples collected only on those days when the flow was no greater than 150 per cent of the current daily average (Table III). These "normal-flow" K values are also shown in Fig-

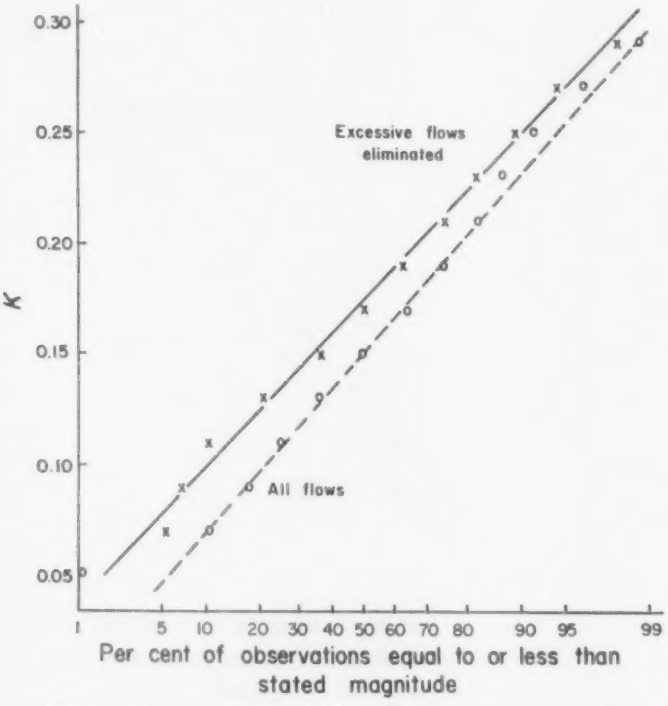

FIGURE 3.—Probability plot of K values for primary effluent.

ure 2. The mean for these selected data was 0.160, with a standard deviation of ± 0.060.

Table IV groups the observed K values for all primary effluent samples in order of magnitude and frequency. These values ranged from a low of 0.055 to a high of 0.290. As shown in Figure 3, the mean was 0.150 and the standard deviation, ± 0.062. To complete the comparison between raw sewage and primary effluent, the grouping in Table V excludes the K values for primary effluent samples taken at times of excessive storm water flows. Under these conditions, the mean value for K was 0.173, with a standard deviation of ± 0.060.

The possibility of a significant difference between K values for weekend and weekday flows was investigated and none was found.

TABLE V.—Grouping of Normal-Flow Primary Effluent Samples

K Value	No. of Observations	Cumulative No. of Observations	Per Cent of $n + 1$ Observations
0.05 (0.040–0.059)	0	0	0
0.07 (0.060–0.079)	3	3	5.3
0.09 (0.080–0.099)	1	4	7.0
0.11 (0.100–0.119)	2	6	10.5
0.13 (0.120–0.139)	6	12	21.1
0.15 (0.140–0.159)	9	21	36.8
0.17 (0.160–0.179)	8	29	50.9
0.19 (0.180–0.199)	7	36	63.1
0.21 (0.200–0.219)	7	43	75.4
0.23 (0.220–0.239)	4	47	82.5
0.25 (0.240–0.259)	4	51	89.5
0.27 (0.260–0.279)	3	54	94.7
0.29 (0.280–0.299)	2	56	98.2

Discussion

Increases in K from the originally accepted 0.1 value have been cited in the literature. Ruchhoft *et al.* (4) and Ruchhoft (5) in 1948 observed velocity constants for sewage ranging from 0.07 to 0.25 with means ranging from 0.16 to 0.20. Rhame (6), by the use of the "two-point method," observed K values for raw sewage ranging from 0.108 to 0.413 with a mean of 0.209.

For further evaluation, the comprehensive review by Orford and Ingram (7) reveals that both K and the ultimate BOD, L, may vary widely according to the time of observation. They point out that K values for normal domestic sewage vary from 0.25 for a 3-day BOD to 0.11 for a 14-day BOD. It is noted that the raw sewage and primary effluent of the Nut Island sewage treatment plant (under conditions of all flows) both have a mean velocity constant approximating 0.15. However, such similarity is not evidenced by simultaneous 24-hr samples of either raw or treated sewage. But since the interest is to determine a revaluated statistical average, an overall 0.15 value is significant.

In his work at Minneapolis-St. Paul, Williams (2) observed an average K for raw sewage to be 0.161. If the data from samples collected during exceptional storm water flows are excluded, then the mean K for Boston raw sewage of 0.160 is close to the Minneapolis-St. Paul figure. A similar comparison for primary effluents, using the data in which exceptional storm flows are excluded, shows a K of 0.173 for the Boston-Nut Island effluent as compared to 0.191 for Minneapolis-St. Paul. Such comparisons are justified since Williams obviously dealt with domestic sewage alone, and there was no mention of combined sewage flows.

Although the K values for raw sewage do range from 0.05 to 0.29, it is noted in Figure 2 that for all flows, 75 per cent of the values are above 0.10, and that 50 per cent of the values can be expected to fall between 0.10 and 0.18. Further, if excessive dilution is to be excluded, then 85 per cent of the K values are above 0.10, and 50 per cent can be expected to fall between 0.12 and 0.20.

A similar comparison may be drawn from Figure 3 in which 80 per cent of the K values for all primary effluents are above 0.10, and 50 per cent

may be expected to fall between 0.11 and 0.19. Excluding excessive dilution, then 90 per cent of the values can be expected to be above 0.10 while 50 per cent will be expected to be between 0.13 and 0.21.

With a K value of 0.15, 82.3 per cent of the ultimate BOD should be satisfied in 5 days rather than 68.4 per cent with a K of 0.10.

Summary

Raw sewage entering the Nut Island sewage treatment plant has a mean K value of 0.140 under conditions of "all flow," and a mean K value of 0.160 under conditions of "normal flow" (excluding days of excessive storm water dilution). Similarly, the primary effluent has a mean K value of 0.150 under "all-flow" conditions, and a mean K value of 0.173 under "normal-flow" conditions.

Acknowledgment

The work herein described was accomplished at and by the Laboratory of the Nut Island sewage treatment plant. Assistance in conducting the many BOD tests was rendered by C. F. Bowman, Assistant Bacteriologist.

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CORRECTIONS FOR TREATMENT DESIGN MANUAL

Two corrections in the FSIWA Manual of Practice No. 8 (ASCE Manual of Engineering Practice No. 36), "Sewage Treatment Plant Design," were reported in the July 1959 Journal (p. 818). Since that time three more corrections have been reported.

1. On page 123, the first word of line 7 should be spelled "sherardized."
2. On page 140, the sentence which immediately precedes Table 18 and which reads, "Values of f for various types of surface are given in Table 18," should be deleted.
3. On page 227, Figure 15, the right-hand ordinate should read as follows: "Btu per hour per capita."

FEEDWELLS FOR DENSITY STABILIZATION

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This paper deals with the problem of density-current short-circuiting in clarification basins. It will be shown how the tendency of new feed to plunge and stratify at the bottom of a basin may be utilized to minimize short-circuiting. The design features necessary to bring about this stabilization of the flow pattern will be discussed. Finally, a feedwell highly effective for the purpose, one which embodies a new arrangement for distributing and stilling the feed flow, will be presented.

Density Currents and Density Stabilization

As a suspension passes through a settling basin it loses settleable solids. Settleable solids are necessarily heavier than the suspending liquid, so by losing them the suspension becomes progressively lighter. Therefore, when new feed is introduced into a basin of partially clarified liquid it tends to plunge to the bottom as a "submerged waterfall" (1) and to spread out as a heavy density layer beneath the lighter supernatant.

Density-Current Velocities

Figure 1, which represents a longitudinal section of a basin, illustrates density stratification. New feed enters at *a*. Because it has a higher specific gravity than the supernatant in the basin, it sinks to the bottom and is deflected to form a "density current" from *b* to *c*.

When the new feed subsides from *a* to *b*, the system loses a certain amount of potential energy. Unless this energy is dissipated against hydraulic losses, it must produce a current velocity, regardless of tank shape, which can be calculated as follows:

$$v = \sqrt{2gh\,\frac{d_f - d_c}{d_f}} \quad \ldots\ldots (1)$$

or

$$v = \sqrt{2ghF\,\frac{d_s - d_c}{d_s}} \quad \ldots\ldots (2)$$

where

v = current velocity,
d_f = density of feed stream,
d_c = density of supernatant,
d_s = density of suspended solids,
F = weight fraction of solids in suspension, and
h = height through which current subsides.

Table I shows the maximum velocities which can be attained as a result of this subsidence, assuming that the

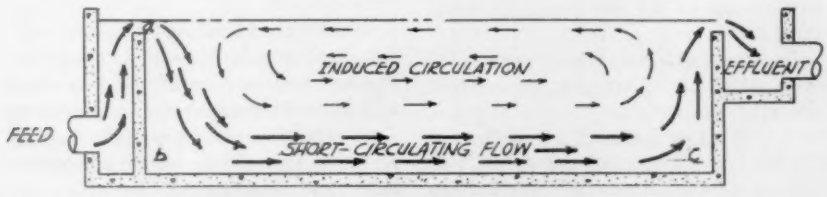

FIGURE 1.—Density currents in a conventional basin.

TABLE I.—Maximum Velocities of Density Currents Attributed to Solids Subsidence

SS (mg/l)	Velocity (fps)	
	h = 10 ft	h = 20 ft
10	0.06	0.09
100	0.20	0.28
1,000	0.64	0.90
10,000	2.0	2.83

solids have a density of 2.65 (silt) and the liquid, 1.0 (water).

The density current so produced traverses the basin and is deflected upward at c against the effluent wall. If the current loses neither solids nor velocity, the kinetic energy it retains at the effluent wall will be exactly sufficient to elevate it to the overflow weir. Of course the current does lose velocity in traversing the basin, but there are two factors which counteract the loss.

First, the density decreases through loss of some of the suspended solids as the current traverses the basin. The kinetics of flocculation reactions (2) (3) are such that the suspension is losing solids at the highest rate during the first part of the sedimentation time. Therefore, when the current arrives at c the density difference between it and the supernatant is greatly diminished. It now takes far less velocity to elevate the current to the overflow weir than was imparted to the current initially by subsidence.

Second, a density current in practice does not derive its velocity solely from subsidence. A substantial part is residual feed-stream velocity which is not dissipated by the feedwell structure.

As a result, even though there is substantial loss of velocity, the current can still short-circuit to the overflow weir. A flow pattern as shown in Figure 1 is set up, and this is the pattern generally seen in clarification basins. Bold arrows indicate the flow through the tank; lighter arrows mark an induced circulation which is not in itself significant in the sedimentation behavior.

Controlling Short-Circuiting

There are two alternative and fundamentally different approaches to preventing formation of such a short-circuiting flow pattern. One is the hydraulically effective "pipe line" approach advocated by Camp (2) (4) and many others. If the basin is made shallow enough, narrow enough, and long enough, the flow tends to traverse the basin with a minimum of dispersion or short-circuiting (Figure 2). The potential instability caused by having higher-density feed-suspension at the influent end of the basin, and lower-density clarified-suspension toward the effluent end, is counteracted by the friction losses along the flow path. Any given slug of feed liquid ideally moves along the basin as a more or less vertical "front." This type of stabilization reaches its ultimate in a pipe line.

While effective hydraulically, this approach is of debatable attractiveness from an engineering or economic standpoint. It has been shown previously (5) that density-current short-circuiting is harmful only where removal of suspended solids is a function of detention time as well as of overflow rate. The most important economic arguments for basins of the rather extreme dimensions necessary for dynamic stabilization are based on the assumption (which the authors believe to be invalid) that detention time is never a direct factor on determining removals.

Whenever detention time and overflow rate are both factors, there will be some combination of the two which will show the greatest over-all economy. This optimum will vary from one application to another, but given reasonable flow stability or detention efficiency in each case, it usually seems to

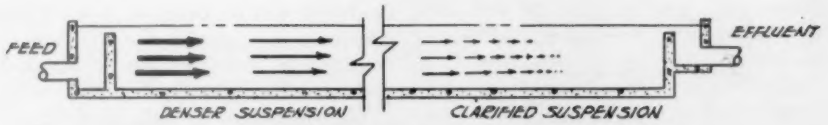

FIGURE 2.—Basin with hydrodynamic stabilization (idealized).

call for basins of substantial depth and appropriately reduced area.

The alternative way of minimizing short-circuiting is to reduce all flow velocities sufficiently so that hydrostatic stratification prevails. Thus in Figure 1 if the velocity at *c* were insufficient to elevate the current up through the layers of supernatant, the stream would tend to fall back. The short-circuit path would be broken, and density-stabilized stratification should result. The basin would approach a hydrostatically stable condition, with clarified supernatant being displaced upward selectively through invasion of new higher-density feed from below. Thickeners and classifiers have long relied on this type of flow stabilization (6) in settling suspensions heavily loaded with solids. Its application to the lightly loaded suspensions common in waste treatment, however, requires special design features.

It will be apparent that short, deep basins are relatively more conducive to density stabilization. In a short basin a density current would cross the bottom to arrive at location *c* more quickly, and would retain a relatively higher specific gravity at that point. This fact, together with the increased height of the effluent wall, would require that it have a relatively greater velocity to carry it up to the overflow weir.

In a deep basin, a density current could potentially gain more velocity by virtue of the greater distance *h* which the new feed has available for plunging, but plunging can be avoided by introducing it at the bottom of the basin. In a dynamically stabilized basin (Figure 3) this would not appear too advantageous because what is desired there is a uniform distribution of new feed from top to bottom. In a density-stabilized basin, on the other hand, bottom-feeding places the new high-density feed precisely where it is wanted.

When a feedwell is used to conduct the new feed to the bottom of the basin, the well has to be properly designed and proportioned. If it is too large, new feed may plunge through clarified layers of supernatant within the feedwell itself, thus nullifying its intended function.

The main difficulty in establishing density stabilization within clarification basins lies in dispersing the velocity of the input stream. When the load of suspended solids is light, the kinetic energy of the feed stream is far more than sufficient to upset density stratification. Unless the energy and momentum of the infeed stream are dissipated or dispersed to a high degree, short-circuiting currents will be formed.

Feedwell Fundamentals

Theoretically there are three ways to cope with the infeed velocity: con-

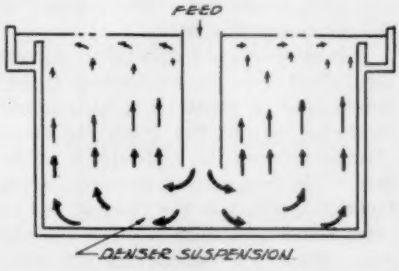

FIGURE 3.—Basin stabilized by density gradient (idealized).

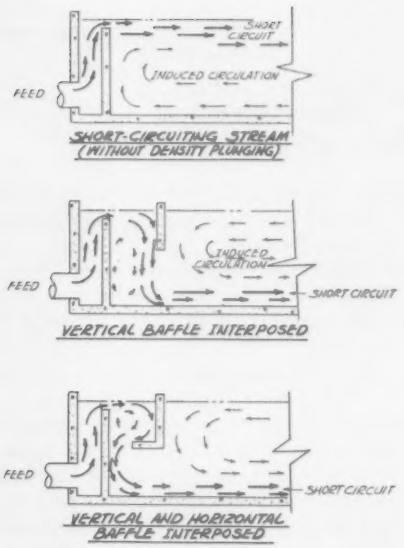

FIGURE 4.—Deflection of short-circuiting stream by baffles.

vert the corresponding energy of the feed stream to head, extract it as work, or dissipate it as hydraulic loss. The first way has not seemed practical because of the large or alternatively complex nozzle systems required. The second expedient is scientifically attractive but also has not seemed practical because of mechanical complexity. Therefore, nearly all feedwell structures are intended to cause irreversible dissipation of the input energy as hydraulic loss. The input energy is dispersed as turbulence, which in turn must decay if it is not to cause agitation and, hence, defeat the purpose of the sedimentation basin.

It is astonishingly difficult to convert the infeed velocity to random turbulences, and to cause these turbulences to decay, within the relatively small volume allotted to a feedwell structure. The feed stream normally issues from its conduit in the form of one or more submerged, relatively low-velocity jets. Such jets persist for an enormous number of diameters. Anyone who has worked on the problem of feedwells will be aware of this. Those who have not may get some idea by considering the persistence of such oceanic currents as the Gulf Stream.

The literature is full of feedwells and feed-baffling arrangements. In general, the expedients in common use may be classified as follows:

1. Stream deflecting baffles,
2. Devices which break up the feed stream into a number of smaller jets or streams, and
3. Impingement of opposed jets or streams.

A feedwell or feed arrangement may often employ more than one of the above expedients in combination. Very few, however, approach the effectiveness necessary to establish density stabilization on lightly loaded feeds.

Deflecting baffles change the direction of a current, and thus can direct it along a longer path to the overflow. They dissipate some part of the current velocity as turbulence, but nowhere near enough. The current, although redirected, perhaps fanned out and somewhat slowed, still retains short-circuiting velocity even after several deflections (Figure 4).

The commonest means used to break up the feed stream are perforated or "picket-fence" baffles. It has been observed that unless these have a relatively small percentage of open area, the current meanders through them without much apparent net effect. The individual jets tend to coalesce down-

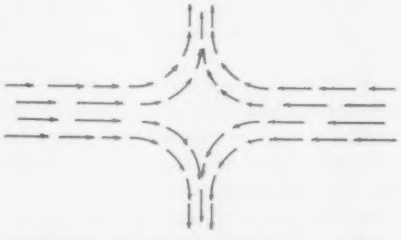

FIGURE 5.—Flow paths for impinging streams.

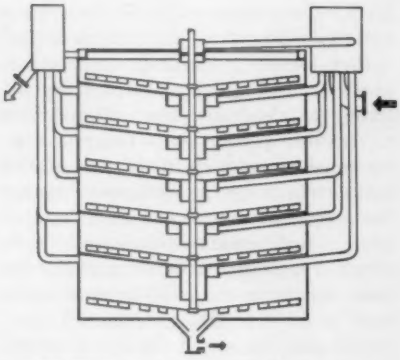

FIGURE 6.—Six-compartment tray thickener.

stream from the baffle and reconstitute the current. If the openings are made narrow and are widely spaced relative to the opening width, the baffle becomes theoretically more effective but practically tends to plug if there is any debris in the flow being treated.

Impinging streams tend to behave as though the individual streams were being diverted by a virtual baffle (Figure 5). Although in general it appears that more turbulence is produced and hence more energy extracted from the streams than if they had hit real baffles, a substantial fraction of the original velocity tends to remain in the redirected currents flowing away from the locus of impingement. This is shown with particular explicitness in a thesis by Barham (7).

Effective Feedwells Using Conventional Expedients

The tangentially fed circular feedwell represents a special case of deflecting baffle, and is of interest because it has proven effective under favorable conditions. In it the feed jet is continuously deflected into a circular or helical path, so that by the time the stream emerges from the bottom it has traveled a substantial distance. If the well is shallow, the stream still maintains its identity. It spirals down inside the well like a snake, and emerges at the bottom apparently somewhat slowed and broadened, but still as a discrete stream directed on a tangent from the bottom of the feedwell toward the periphery of the basin. If, on the other hand, the feedwell is deep, the infeed jet must travel a relatively long helical path down through the feedwell, and this long path in continuous contact with the stationary feedwell wall serves to disperse and dissipate the current to a significant extent. The flow from the bottom of the well still retains some tangential velocity, and, hence, the infeed energy is not completely dispersed, but there can be quite uniform radial distribution of the flow. This is demonstrated by the following example.

Some years ago an installation of flue-dust thickeners designed as shown in Figure 6 was giving operating trouble because of a severe scaling problem. It was believed that conversion of the units for what has been termed "selective density feeding" would be justified to ameliorate the scaling problem, and that overflow clarity would not suffer as much as formerly believed.

The trays were removed, and large-diameter, deep, tangentially fed feedwells were installed as shown in Figure 7. Removal of the trays increased the overflow rate by a factor of 6, and

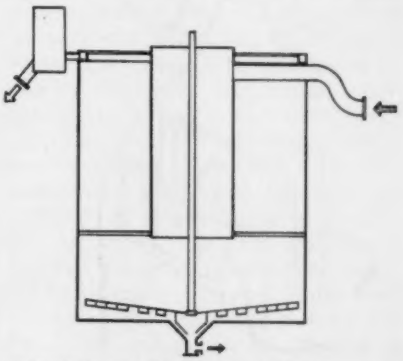

FIGURE 7.—Converted single-compartment thickener.

according to the overflow-rate clarification theories widely accepted at that time this change should have decreased the settling capacity of the units by the same factor. The change, however, led to density-stabilized flow patterns which yielded detention efficiencies in excess of 80 per cent, as compared to about 15 per cent for that particular tray design. The gain in detention efficiency more than offset any efficiency losses from the concomitant increase in overflow rate (5) and the net result was a substantial improvement in overflow clarity.

Where detention efficiencies above 80 per cent can be attained with feedwell structures as simple and foolproof as that shown in Figure 7, the economic advantage for more complicated structures becomes a matter for careful investigation. The designer almost always has the alternative of gaining the desired "effective detention" (3) by increasing the size of the unit. An increase of only 8 per cent in the linear dimensions of a settling basin will increase the effective detention (other factors remaining constant) as much as improving the detention efficiency of the basin from 80 per cent to absolute perfection. Therefore, after considering both initial installation and potential operating inconveniences, the costs of any more-effective but more-complicated structure have to be weighed against those of making the basin larger by the small amount necessary to give equal performance.

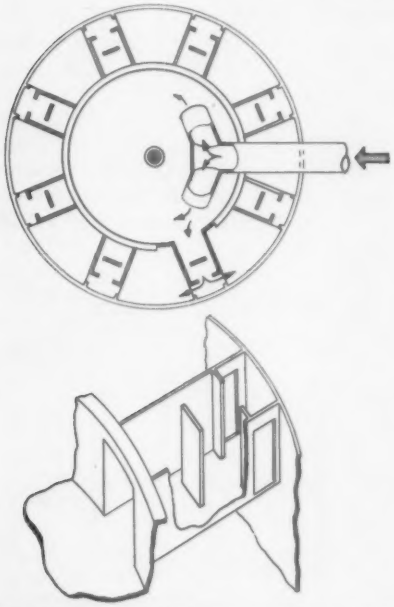

FIGURE 8.—Radial-type velocity-dispersing feedwell.

The case described above, of course, was particularly favorable for density stabilization. First, the specific gravity of flue-dust scrubber water containing about 4,300 mg/l of heavy iron oxide is much higher, relative to clarified supernatant, than is common in treatment of waters or wastes. Therefore, it shows an unusually high tendency for density stabilization. Second, the units as converted were unusually deep and had a high depth-to-diameter ratio. Both are favorable to density stabilization. Subsequently, it has been found that units of optimum economic design would usually be of lesser depth, if a high detention efficiency can be maintained.

In shallower units, the feedwells are not as deep and, hence, the infeed velocity is not as completely dispersed. With lighter feeds the residual tangential components at the bottom of the feedwell are enough to upset density stratification. Therefore, more effective means of dispersing the infeed velocity are usually required.

It was conceived that a feedwell should contain some velocity-dispersing element for converting the infeed velocity to sharp, small-scale turbulence which would decay quickly, and that it should leave no residual tangential components at the bottom of the well. It should also comprise a volume in which the turbulence is given time to decay before the flow is released to the settling basin proper. A great many known feedwell structures were

considered, and many were tested in a 12-ft diameter pilot clarifier. One was highly effective.

The feedwell structure (4) shown in Figure 8 comprises a series of ducts radiating outward from a central infeed enclosure, like spokes of a wheel. On each side of each radial duct, near its outer end, is a vertical slot through which feed emerges tangentially. The feed is thus broken up into a series of tangentially directed, narrow, submerged jets. Half the jets are directed clockwise, and the other half, counterclockwise. Each clockwise jet impinges on a counterclockwise jet from an adjacent radial duct. The spacing of the slots precludes coalescing of the streams, while the narrowness of the jets together with their impingement leads to a rapid conversion of their velocity to random turbulence. The balance between clockwise and counterclockwise streams cancels any net or residual tangential components of velocity.

The velocity-dispersing element described above was surrounded by a feedwell proper which confined the turbulence giving it a chance to decay, and which delivered the flow to the lower part of the clarifier basin. The arrangement of a pilot clarifier as well as the test results are summarized in Table II.

As can be seen, the feedwell gave a high "standard detention efficiency" in a basin with a depth-to-diameter ratio approximating those commonly encountered in the field. Standard detention efficiency is a value calculated from dye tests (3) and is equal to:

$$\frac{\int_0^\infty D dt}{t_N \int_0^\infty \frac{D dt}{t}} \quad \ldots\ldots\ldots (3)$$

where

D = dye concentration in overflow,
t = time after introducing slug of dye into feed, and
t_N = nominal detention time of basin.

TABLE II.—Pilot Clarifier Data

Item	Quantity
Clarifier diam	11 ft 7 in.
Sidewall depth	4 ft 6 in.
Feedwell diam	30½ in.
Feed suspension*	986 mg/l
Flow rate	99 gpm
Standard detention efficiency	91.5%

* Precipitated $CaCO_3$.

Standard detention efficiency has been found to correspond reasonably closely to the actual detention efficiency of a basin as determined from sedimentation results.

The feedwell is functionally effective and has been installed in a number of basins. However, as with all feedwell structures which depend on passing the flow through narrow slots, tubes, or orifices, it is susceptible to plugging with debris. An ideal feedwell should have no closed ducts or pluggable constrictions.

New Type of Feedwell

It appears functionally necessary to disperse the infeed velocity rapidly and completely as small-scale turbulence in the feedwell if density stabilization in the associated basin is to be attained. At the same time some experienced engineers have steadfastly rejected as undesirable from an operating standpoint all structures having multiple slots, ducts, orifices, baffles, pickets, or other conventional means of effecting such dispersion. In order to resolve the dilemma a new means of velocity dispersion was finally developed as shown in Figure 9 (8).

Within this new feedwell structure are two adjacent, inwardly open, submerged races or channels. Half the feed is introduced tangentially into Channel *A*. It follows this race around the feedwell and although held in the channel by centrifugal force it is displaced inward by the continuing flow of new feed. The other half of the flow is introduced tangentially into Race *B* in the opposite direction to the flow in

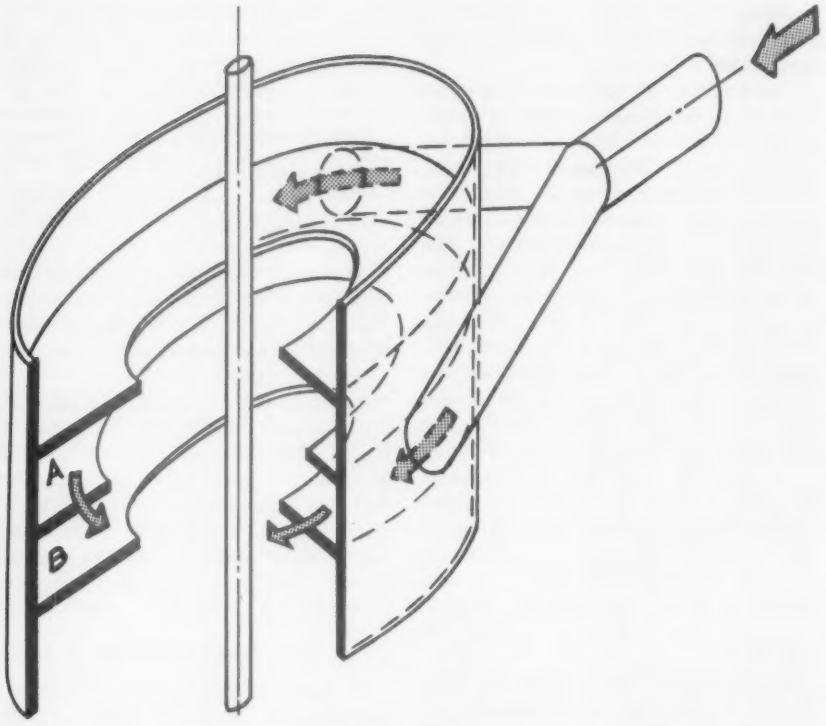

FIGURE 9.—Tangentially fed velocity-dispersing feedwell.

Channel *A*. As they are displaced inward beyond the protection of the raceways, these two oppositely directed streams or currents are compelled to shear one on the other along their entire length. In the zone of shear the velocities are instantaneously dissipated as turbulence. Since the streams are equal and oppositely directed there are no residual tangential velocity components.

It will be apparent that the structure is simple. It offers no apparent possibility for plugging since the raceways are completely open to the inside and present no upstream edges to catch debris. There is only one split of the entire feed stream. From a practical operating standpoint the structure has been completely satisfactory.

Functionally, the structure has given detention efficiencies as high as any other tested. Under almost identical conditions to those for the pilot clarifier described previously, the new feedwell gave a standard detention efficiency of over 90 per cent.

The diffusing structure actually is more effective in distributing the flow around the feedwell and eliminating residual velocity streams than was anticipated. As originally conceived, the races were made in the form of volutes, thus forcing the feed streams by channel geometry to "overflow" progressively inward and beyond the protecting walls of the guide channels or raceways. However, the completely circular raceway shown in Figure 9 is apparently just as effective in distributing the flow. Although of unusual proportions, it will be recognized that the flow in a raceway constitutes a

free vortex, somewhat analogous to a liquid cyclone. It is characteristic of such flow patterns that higher energy elements of the flow stratify near the periphery of the vortex. As elements of the flow lose energy due to fluid friction they are selectively displaced inward. There develops an energy stratification with a gradient toward the axis of rotation which is uniform around the raceway, and as a consequence the inward displacement or radial components of flow also tend to be uniform around it (9).

The two velocity streams, with no escape or bypassing of any element of either stream, are compelled to interact completely by the following expedient: Figure 10 shows a cross section of the two raceways or channels, with a stream circulating in one direction in the upper channel, in the opposite direction in the lower one. The boundary wall or member *a* which separates the two streams is made narrower than the outside walls *b* so that it does not project as far toward the axis of rotation of the streams. As long as the circulating streams in the raceways retain tangential velocity, there is a centrifugal force tending to hold them in the raceway. As the two streams are displaced inward toward the axis of rotation, they pass first beyond the separating member *a* and shear on one another with a velocity differential equal to twice the speed of either stream. In this shear zone turbulence vortices are immediately formed, and the gross tangential flow components of the two streams are neutralized. The resulting mass of turbulent fluid then no longer exerts centrifugal force and is preferentially displaced inward past the boundary walls by elements of the flow which have not lost their tangential velocity. As a result, the flows stay within side members *b* until after they shear and lose their velocity. All of the flow is forced to pass through the shear zone. Thus it is an inherent characteristic of the structure that dispersion of the infeed velocity will be complete.

Usually the two channels of the velocity-dispersing structure in a feedwell are placed as close together as possible. In the above illustration they have a common bounding member, *a*. The closeness of the two shearing

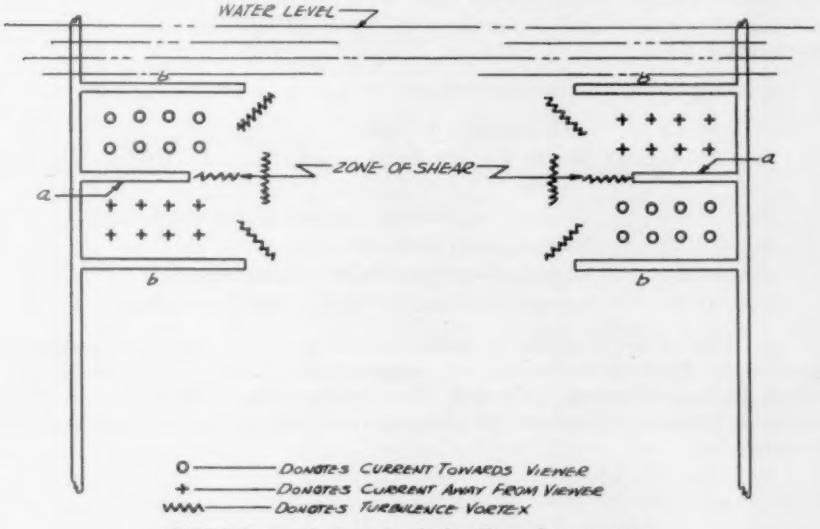

FIGURE 10.—Action of velocity-dispersing structure.

streams, together with the doubled differential velocity, gives the maximum possible velocity gradient in the zone of shear. This high velocity gradient gives a rapid conversion to the fine-grained or small-scale turbulence which tends to decay most quickly.

If for any reason it is desired to create a larger-scale, more-persistent turbulence, it can be done by separating the raceways somewhat, thereby reducing the velocity gradient in the shear zone.

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USPHS TRAINING COURSES

The Public Health Service's Robert A. Taft Sanitary Engineering Center is continuing the opportunity for training in various specialized courses during the winter season. These courses cover radiological significance of water and air pollution, water quality management, water bacteriology, and radiological materials in food.

The listing of the remaining courses is:

Feb. 15–19	Radionuclides in Water
Feb. 29–Mar. 11	Water Quality Management—Sanitary Engineering Aspects
Mar. 14–25	Sanitary Engineering Aspects of Nuclear Energy
Apr. 18–29	Radionuclides in Foods
May 9–20	Organic Industrial Wastes Characterization
May 23–27	Inorganic Industrial Wastes Characterization

These courses are available to those interested in the particular programs mentioned. There is no tuition fee. Applications should be addressed to the Chief, Training Program, Robert A. Taft Sanitary Engineering Center, 4676 Columbia Parkway, Cincinnati 26, Ohio, or to a Public Health Service Regional Director.

Industrial Wastes

JOINT INDUSTRY AND CITY WASTE TREATMENT: COMBINED APPROACH AS VIEWED BY INDUSTRY *

By George P. Schrader

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The Kanawha River Valley

The City of South Charleston, West Virginia, and the South Charleston Plant of Union Carbide Chemicals Company are located on the Kanawha River about 50 miles from the junction of the Kanawha with the Ohio River. The Kanawha River is formed by the junction of the New and Gauley Rivers at Gauley Bridge and runs 90 miles generally to the north. Two major tributaries are the Elk River which enters at Charleston, and the Coal River which enters at St. Albans.

The Kanawha River Valley is narrow and winding throughout its length and its industrial and residential development is congested to the south and east of Nitro. The level areas on one bank of the river usually face a sheer cliff or steep hills on the opposite bank. Through the most heavily industrialized section of the valley run the mainline of the Chesapeake and Ohio Railroad and a branchline of the New York Central. Two highways also compete for space along the waterway. To the north and west of Nitro the river bottoms are wider and can be made available to industry.

Visitors entering the valley on U. S. Route 60 at night for the first time are amazed at the scope of the industrial operations. From the glare of the furnaces at Alloy to the always-lighted units of the chemicals plants at Nitro, visitors may also see industries producing metals, power, glass, tools, chemicals, ordnance, machinery, and synthetics.

Manpower for such industries is derived from the adjacent communities as follows:

Town	Population
Smithers	2,200
Montgomery	3,500
Cedar Grove	1,700
Belle	4,000
Charleston	90,000–100,000
South Charleston	21,000
Dunbar	8,000
St. Albans	10,000
Nitro	3,300

Figure 1 reveals that the major portions of the population and the industry contributing to any problem or project are located in the upper half of the valley.

The Industry Problem

Union Carbide Chemicals Company makes nearly 400 chemicals, for thousands of uses, from natural gas, salt, air, and water. Only two or three are prepared directly for retail use. The rest are sold to other chemicals companies or processors for preparing the

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

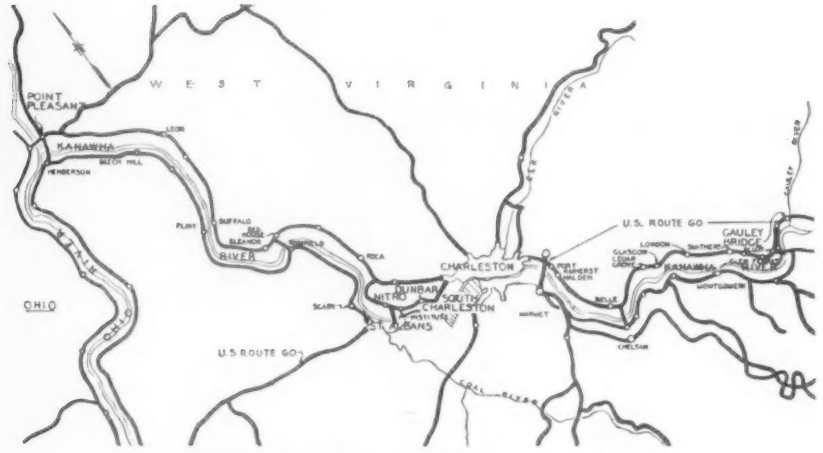

FIGURE 1.—Map of the Kanawha River Valley.

finished products. The residues from the manufacturing processes for these chemicals make up Carbide's waterborne pollution abatement problem.

The first question usually asked is, "What chemicals compose the industrial wastes?" The answer must be that it is not known, since it is difficult to identify organic chemicals in a waste stream. In addition, some of the processes are proprietary and the Company does not wish to reveal the nature of the residues or liquid wastes.

The problem of this particular chemicals industry is probably familiar in that it is caused partially by an unpredictable growth. Figure 2 is a map of South Charleston. The Chemicals Company in 1923 started its South Charleston Works by leasing the site of some existing buildings (Area *A*) built during the First World War by the Rollins and the Klipstein Chemicals Companies. In 1927 the expansion began with the purchase of Blaine Island. Where possible, existing facilities were used, replaced, or expanded. Cooling water was returned to the river after single stage use. Occasionally, heat exchangers leaked and contaminated the product or the cooling water. It was often necessary to clean process equipment and drain off the wastes. During this period, no attempt was made to segregate clean water from contaminated water.

As the Federal Government and the various state governments became interested in the problems of water use and pollution, a number of regulatory agencies were created. A very prominent one is ORSANCO, or the Ohio River Valley Water Sanitation Commission, representing Illinois, Indiana, Kentucky, New York, Ohio, Pennsylvania, Virginia, and West Virginia. With the creation of ORSANCO, municipalities and industries could foresee a time when they must expect to cease polluting the streams.

Carbide had a threefold problem: (*a*) to collect and segregate the wastes, (*b*) to determine the flow and BOD load, and (*c*) to determine the method of treatment. Carbide's South Charleston installations are physically divided into five separate areas by the Kanawha River and intervening plants or residential areas in the city, as shown in Figure 2. The Lower Mainland includes the previously mentioned Area *A* and is separated from the other areas by the back channel and the Columbia Southern Chemical Com-

pany plant. Blaine Island, Area *B*, is separated from the rest of the plant by the river. Area *D* in North Charleston is across the river from Blaine Island. The Upper Mainland, Area *C*, is east of the Columbia Southern plant and across the back channel from Blaine Island. The Technical Center, Area *E*, is located two miles from the rest of the plant and is 200 ft higher.

In 1952 there were at least 60 outfalls from the chemicals plant into the back channel. Measurement of waste flow was difficult since the wastes were not yet segregated from the cooling water sewers. The Pollution Abatement Department made a survey in which the unit supervisors estimated the flows. The unit flows were totaled to give an estimated waste flow from the plant.

Segregation and collection of water-borne wastes were started on Blaine Island in 1953. Weirs, manholes, sewers, flumes, and a circular concrete sump were constructed to collect an estimated 37 per cent of the total contaminated flow and 61 per cent of the total BOD load. Upon completion of this system, flow measurements were made of the wastes passing through the sump to the river.

In 1955, the Pollution Abatement Department made a new survey for the entire plant. Based on a comparison between the estimated flow from units discharging into the collection system and the actual flow through the sump, a new design flow was established.

In April 1954, a back channel survey was made to establish the BOD load discharged to the river. Since it was impractical to measure the load from each contributing unit, the decision was made to determine the BOD at the upstream and downstream ends of the back channel. The difference in load between the two points was specified as the load contributed to the river by the chemicals plant. A deduction was made to correct for Westvaco's wastes.

During the summer months of the year Carbide uses 190,000 gpm of cooling water. Of this, it is estimated that 5,600 to 11,000 gpm are loaded with 130,000 lb of 10-day BOD per day.

Industry's Developmental Work

As a basic assignment it was necessary to determine which wastes could

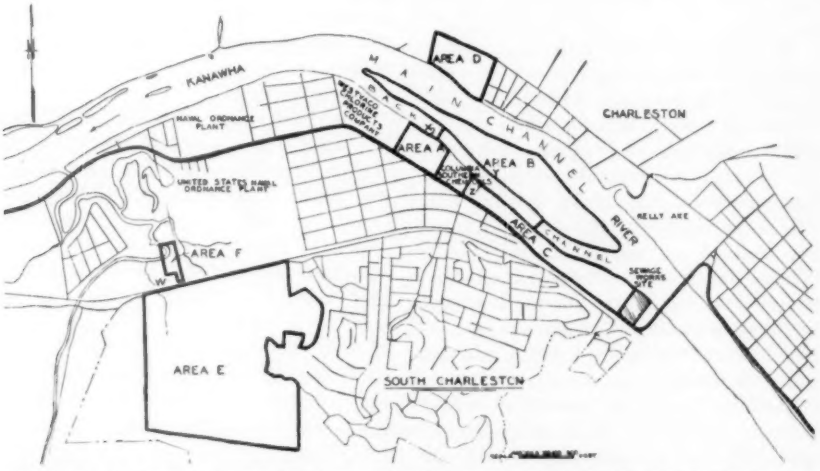

FIGURE 2.—Map of South Charleston showing the areas of significance in the report.

be treated and which method of treatment was best.

A confusing discrepancy was present because of the fact that the industry's computed total pollution loads were always lower than those found in the river by the regulatory agency's stream surveys. Cooperation with the agency soon proved that the differences were not due to lack of suitable equipment or improper techniques. Search of the literature revealed there was practically no information available on the BOD values of synthetic organic chemical compounds such as those which would probably be present in the waste streams from the manufacturing processes. A program of experimental work (1) was undertaken to determine whether these chemicals would decompose biochemically at the same rate as domestic sewage and to reconcile these differences. This work revealed the following facts:

1. Synthetic organic chemicals are not attacked biochemically in the same manner as sanitary sewage, when present in ordinary detectable amounts.
2. It may be necessary to run the BOD determination for more than five days in order to estimate the possible pollution by synthetic organic chemicals.

Studies were undertaken to determine the effect of specific organic materials on a normally functioning biological oxidation system, when the organic materials were added intermittently at high instantaneous concentrations to simulate shock loading (2). The experimental data were obtained by feeding a replica of the South Charleston process effluents to the dispersed seed aeration system, reported by Heukelekian (3) as the non-flocculent growths method. Further studies were undertaken to determine if acclimatized bacterial growths would improve the biochemical oxidation of synthetic organic materials of interest to Carbide (4). The materials investigated were known to be difficult to oxidize in a non-acclimated culture. Successful acclimation of microorganisms from Kanawha River water was obtained for all of the organic chemicals studied.

In 1954, a large-scale pilot plant was constructed to collect data for determining final design and preparing final estimates. The pilot plant was located so that the wastes collected from Blaine Island could be the basis for experimental development of treatment methods. Briefly, the equipment consisted of: (*a*) 1 140-gpm primary pump, (*b*) 2 6-ft diameter primary clarifiers, (*c*) 2 1,300-gal mixing tanks, (*d*) 1 600-gpm recirculation pump, (*e*) 1 6-ft diameter secondary clarifier, (*f*) 1 12,000-gal storage tank, (*g*) 1 4- by 4- by 22-ft long aeration tank, (*h*) 1 3-gpm sludge recirculation pump, (*i*) 1 5-ft diameter final clarifier, and (*j*) necessary instruments, feeders, and accessories.

Several runs were made in 1955 and 1956 to establish design criteria and bases for evaluation of different processes. These runs used conventional activated sludge, the biosorption process, trickling filters, and various combinations of these. Both stone media and Dowpac HCS * were evaluated in a trickling filter. Nutrients, temperature, sludge disposal, and pH adjustment were all studied during the pilot plant operation.

The pilot plant studies established the optimum operating condition for each process on which the later design was based. The choice of process was based on the consulting engineer's economic evaluation of the processes, covering the initial cost of construction, annual operating costs, and the ease with which each process would fit into the State authorities' future requirements for expansion. The activated sludge process was recommended and accepted. Part II of this paper

* Manufactured by Dow Chemical Co., Midland, Mich.

will discuss reasons for this recommendation.

While the pilot plant was in operation, samples of various construction materials were placed in the top and bottom of the trickling filter, in the aeration tank, secondary clarifier, final clarifier, and raw waste sump to determine which metals and coatings would be satisfactory. In addition, the Company in 1954 made a study of subsurface disposal of water-borne pollution at the South Charleston plant. The method was reported as undesirable for several reasons, the chief of which is that pretreatment of wastes for underground disposal is a critical and expensive process. One other investigation reached a similar conclusion (5).

Industrial Approach to Problem

During the period 1952 to 1954, Carbide proposed to design and build its treatment plant over a five-year period. The treatment plant was to be located in the chemicals plant with the primary clarifier and transfer pump station on Blaine Island near the collection sump. All secondary treatment and sludge disposal facilities were to be located on the North Charleston site, Area *D*.

Combined Approach

The City of South Charleston, during the period 1948 to 1954, was designing its own sewage treatment system. For this purpose, Carbide donated the site, Area *F*, to the City.

In July 1954, it was suggested by the City of South Charleston that Carbide install a waste treatment plant large enough to handle the combined wastes of the Chemicals Company and of the City. The reasoning for such a combined plant may be summarized here as:

1. A combined plant would require less initial capital investment than two separate plants. This would result from more efficient use of land, elimination of duplicated equipment, and lower engineering costs.

2. A combined plant would have lower operating costs than two separate plants since duplication of personnel and purchases would be eliminated. Utilities were estimated to be available at lower rates.

3. Admixture of the City's domestic wastes were expected to reduce Carbide's waste nutrient requirements, in particular, nitrogen and phosphorus.

Some negotiation would be required to distribute equitably the initial capital savings and the savings in operating costs. The Company investigated the City's proposal of a combined plant and concluded that it would be feasible and economical to design a combined plant large enough to treat the wastes from the City and Carbide. Its report indicated a capital savings of $150,000 and annual savings in operating costs of $50,000 to both parties.

At this time negotiations were begun with the City about the combined waste treatment plant. The negotiation period required 15 months. To account for this time period, any venture such as this requires approval by the Sanitary Board, the City Council, the bondholders or bond companies, Chemicals Company Management at South Charleston, and Corporation Management at New York. Each reviewing or approving unit had its own lawyer or law department to review the proposal.

During this period of negotiation, these points had to be settled:

1. Ownership of the plant. A City-owned plant would be tax exempt. City ownership would relieve Carbide of acting as a public utility and would prevent the legal difficulties arising from other towns requesting that Carbide treat their wastes.

2. If City-owned, Carbide felt it necessary to be assured that the plant

design would be adequate to handle the industry's wastes.

3. If City-owned, Carbide also felt it necessary to be assured that operation of the works would be separated from political appointments.

The negotiations were completed when three joint-venture contracts were signed on March 15, 1956. These contracts covered the following arrangements:

1. The City will own the sewage treatment works and would accept Carbide's wastewaters for treatment.
2. Carbide will deliver its wastes to the City for treatment and will guarantee not to build a competitive treatment plant.
3. Carbide will furnish pilot plant data for the design basis.
4. Carbide will be responsible for design of the works, with authority to sublet the design if necessary.
5. The City hired the newly formed South Charleston Sewage Treatment Company, a subsidiary of Union Carbide Corporation, to operate the City's sewage treatment works. The Sewage Treatment Company reserved the right to control employment.
6. Financial details were specified, such as sharing costs and setting up rates.

Role of the Water Commission

The West Virginia State Water Resources Commission has had a part in this problem through the years. As early as 1941, the organization, then known as the West Virginia Water Commission, made an effort to ascertain the quantity of pollution being discharged, and requested information as to whether methods of abatement were being considered. However, the war years intervened and little was done until February 1946, when a conference was called to discuss the industrial pollution problem, at which Carbide was ordered to make a thorough survey of all of its discharges and to advise the Commission regularly of progress toward abatement.

In 1954, the Commission in separate letters to Carbide and the City ordered each of them to cease polluting the waters of Kanawha River. The State Water Resources Commission did not specify a detailed criterion on which to base the design of a sewage treatment works. As the result of a conference in February 1957 with the Commission and representatives from the State Health Department, agreement was reached for a tentative design criterion of 42-per cent BOD removal. Later conferences confirmed this agreement.

Because of the rapidly increasing estimates of final cost, in 1958, Carbide requested the engineers to delay design until a resurvey could be made to determine the possibility of reducing the flow, BOD load, and cost of the treatment plant. The State Water Resources Commission requested a report on the progress and a time schedule for future planning, financing, and construction. At its meeting on January 29, 1959, the Commission accepted Carbide's report, approved the time schedule, and requested quarterly reports on design and construction beginning April 1, 1959.

Education and Public Relations Efforts

Prior to the entry of the City into Carbide's pollution abatement program, Carbide made numerous efforts to keep the press and the public informed of its program to reduce pollution of the river. An example was a report by Jenkins in 1954 (6).

The negotiations between the joint-venture partners were confidential until all the details were settled and the contracts signed. On March 16, 1956, the Charleston Gazette carried a story entitled "Carbide Joins City of South Charleston to Build Sewer Plant, Unique Municipal-Industry Co-op Will Operate Works." This front-page

story reported the City Council signing the contracts and elaborated on details of the venture. Since that time Carbide and the City have made a sincere effort to keep the public informed of progress.

Acknowledgments

The background of the joint venture described herein is a review of the actual events as described in the published literature and the private communications of Union Carbide Chemicals Company. The assistance of Messrs. F. H. Belden, R. A. Conway, R. C. Hieronymus, and N. H. Ketcham of Carbide in making available their files is gratefully acknowledged. The encouragement and help of Messrs. L. J. Bowditch, H. W. Hetzer, P. C. Vickers, and F. B. Wotiz of Carbide are sincerely appreciated.

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1960 NUCLEAR CONGRESS

The 1960 Nuclear Congress, consisting of the Sixth Nuclear Engineering and Science Conference, the Eighth NICB Atomic Energy in Industry Conference, and the Sixth International Atomic Exposition, will be held in the New York Coliseum April 4–7. The Federation as one of 28 sponsors is responsible for two papers of a four-paper session on water supply and waste disposal. These are:

"Use of Tritium as a Tracer in Evaluating Waste Discharges," by Warren J. Kaufman and Richard M. Hours.

"Hanford Reactor Effluent Contribution to Environmental Radiation Dose," by Robert L. Jenkins.

A pocket-sized leaflet describing the attractions of the meeting is available to those making a request to the Federation office.

JOINT INDUSTRY AND CITY WASTE TREATMENT: COLLECTION SYSTEM AND TREATMENT WORKS *

By James K. Latham and J. Wiley Finney, Jr.

Partners, Howard K. Bell, Consulting Engineers, Lexington, Ky.

The Collection System

Industrial Wastes

Union Carbide Chemicals Company's wastes originate from four areas, namely, Lower Mainland, Blaine Island, Upper Mainland, and Technical Center. These wastes are to be collected and transported to the treatment works site in the following manner:

1. Lower Mainland. Wastes will be collected in a sump from which they will be pumped through a force main to the western end of an open flume, which flows eastward along the south shore of the Kanawha River Back Channel.

2. Blaine Island. Wastes will be collected in a sump from which they will be pumped through a submerged force main to the western end of the flume described in Item 1.

3. Upper Mainland. Wastes will be discharged into the above-described flume which flume discharges all of the above wastes to the main lift station for delivery to the industrial side of the treatment works.

4. General. The volume of waste delivered to the main lift station will vary from 5,600 to 11,000 gpm, with an average flow of 7,000 gpm. Pumps at the main lift station will be duplicate, variable speed, horizontal, dry pit type units, of special alloy metals construction, to withstand corrosive characteristics of the waste to be handled. Because of occasional "gunky" content of the liquid to be handled, the only valves in pump-piping will be sluice gates on suction lines. Since only one pump will be on the line at one time, a purge water pump will be provided to purge either pump, in the event of shutdown.

5. Technical Center. Waste will be injected into the City's force main sewer along the Chesapeake and Ohio Railroad tracks by hydrostatic head, through the medium of a liquid-holding manhole or stack.

City Wastes

The treatment works is located at the eastern boundary of the City whereas the Kanawha River flows westward. This site for the treatment plant was selected for three major reasons, namely:

1. Least over-all first cost.
2. Confinement of highly corrosive, contaminated wastes within the industrial area along the Kanawha River.
3. Economical power for pumping costs.

The area from which the City wastes are to be collected is high along the south bank of the Kanawha River (Elevation 605 ±), slightly lower in the comparatively flat area between the river and the Kanawha and James River Turnpike (Elevation 600 ±), to rather steep sloped hills to the south of the turnpike. This general lay of the land is broken up by Joplin's

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

Branch, Wards Branch, and Davis Creek, all flowing from south to north and emptying into the Kanawha River. It is therefore apparent that extensive pumping of City waste is mandatory.

Waste from the City will be delivered to the treatment plant site by utilizing eight pumping stations, appropriate force mains, and a number of gravity sewers.

Historical

The program of improvements herein outlined has been designated as Phase 2 of the over-all sewage works program. Phase 1, already complete, consisted of the construction of relief storm sewers in certain portions of the level area of the City, at a total project cost of $2 million.

General

Where existing combined sewers are collected into the system, separation structures will be provided. Five pump stations will be wet pit type with duplicate vertical pumping units on sequence control. Two stations will be dry type vertical pumping units, each having three pumps on program control. The remaining pump stations will be a dry type with duplicate variable speed horizontal pumps, only one of which will be on the line at a time. Delivery from the latter station will vary from a minimum of 2,200 to a maximum of 5,400 gpm, with an average of 4,000 gpm.

Treatment Required

The West Virginia Water Resources Commission had indicated that primary treatment of the City's waste was required, but a fixed degree of treatment or pollution reduction by the industry had not been ordered. However, after conferences with the State Water Resources Commission and the Health Department, the initial facilities are being designed to provide an over-all BOD reduction of 42 per cent. This will be accomplished by primary treatment of the wastes plus secondary treatment of one-third of the total waste volume.

TABLE I.—Design Criteria for South Charleston Domestic Waste

Criterion	Amount	
	Separate Sewers	Combined Sewers
BOD (lb/day/cap)	0.16	0.24
Suspended solids (lb/day/cap)	0.20	0.27

Provisions are incorporated in the plant design to add duplicate secondary units in the future to treat another one-third of the flow and this will increase the over-all BOD reduction to 67 per cent. Ultimate treatment of the total flow in the secondary units will provide for a reduction in BOD of about 92 per cent.

South Charleston City Wastes

The waste in the South Charleston system is essentially domestic and commercial. Flow measurements and analyses of the sewage did not indicate any unusual conditions or problems relating to strength or flow. Conventional design criteria for BOD and suspended solids loadings were used. These are shown in Table I.

The average daily flow for plant design was calculated on the 1985 estimated population at 100 gpd/cap. Design domestic flow is 4,050 gpm or 5.8 mgd, which includes 1,000 gpm flow from Carbide Technical Center. The design population is 42,500 for the year 1985.

Industrial Waste Design

General

The industrial waste flow, averaging 7,000 gpm, will contain an average of 130,000 lb 10-day BOD per day. Peak daily BOD may be double the average.

The pilot plant studies of the industrial waste, which were discussed in Part I of this paper, disclosed that secondary biological treatment of the waste either by trickling filters or by the activated sludge process could be accomplished with reasonable success, if the pH of the waste was controlled between 6.0 and 9.0. With a 6-hr aeration period, the activated sludge process gave a BOD reduction of 90 per cent of the BOD applied to the secondary system. The trickling filter study indicated that the maximum removable BOD by this means was 80 per cent of the total BOD applied and the reasonably economical removal was about 65 per cent at 10°C (recirculation ratio, 8:1).

Economic studies of the two secondary processes indicated the initial construction cost was strongly in favor of the activated sludge process, regardless of the degree of treatment provided. The operation cost of the trickling filter plant was estimated to be more expensive than the activated sludge plant when treatment was designed for 42 per cent and also 65 per cent over-all reduction. This unusual condition is brought about by the fact that the trickling filter designs considered required the treatment of the total flow through the trickling filters and thus the pH adjustment of the total industrial waste flow; whereas, in the activated sludge process, it would be necessary to treat only a portion of the total flow and the resulting reagent costs for pH adjustment would be appreciably less.

The pilot plant studies of the industrial waste indicated two major problems: (*a*) pH and (*b*) primary sludge handling. The pH of the waste may vary from 1.0 to 13.0, with changes as fast as one pH unit in 10 seconds. The sludge was found to have variable characteristics. Leaf tests indicated that vacuum filtration was satisfactory for dewatering the sludge part of the time, but on occasions, the sludge was oily or tacky and the filter media was quickly blinded. The sludge can be compacted by several hours settling.

pH

In order for the secondary biological process to function properly, the pH of the waste must be adjusted to the range of 6.0 to 9.0. It would be desirable to provide lagoons for the raw waste in which the slugs of acid waste and the slugs of alkali waste could be used to neutralize each other, but there is simply no land in the area available for such a purpose. The problem, then, is to provide equipment and control facilities for continuously adjusting the pH to the desired range.

Only the waste receiving secondary treatment will require pH adjustment and since only one-third of the flow will be treated in the secondary process in the initial phase, the pH control facilities will be located immediately ahead of the activated sludge process.

Because of the wide variation in pH, the waste is extremely aggressive to the usual construction materials. Adequate protection must be provided for concrete and steel and special materials must be used which will resist the corrosive nature of the waste. The solvent concentration in the waste stream has added to the problem of selecting suitable coatings.

The industry made studies of numerous coating materials, plastics, and metals by immersing them in the waste stream for extended periods. As a result of these studies, it has been determined that certain epoxy resins and certain phenolic resins will provide adequate protection to concrete and steel surfaces. Industrial waste pumps will be constructed of Hastelloy alloy F.

Primary Sludge

In view of the fact that the laboratory studies did not arrive at a suitable means for dewatering the industrial primary sludge and that the total col-

lection of the wastes will not be completed until the treatment plant and pumping station are constructed, it was decided to defer the planning and construction of sludge handling and disposal facilities until further study could be made after the plant is placed in operation. Space will be reserved for construction of sludge thickeners, vacuum filters, and incinerators, if such methods are proved to be suitable.

Indications are that prolonged settling will concentrate the sludge to a thickness that will permit incineration. Since the industrial sludge is biostatic and may be held for long periods of time, it is desirable to keep the domestic sludge and industrial sludge separate. This will dictate the use of separate primary clarifiers for industrial waste and domestic sewage.

Activated Sludge

Normally, two approaches may be taken to the design of aeration basins for the activated sludge process: (*a*) the basin may be designed on a flow-through principle with a minimum of end-to-end mixing, or (*b*) it may be designed for complete mixing. Carbide pointed out the following advantages of the complete mixing principle when applied to the treatment of an industrial waste:

1. The wastes are diluted by the entire volume of the tank, which in addition to dilution, provides equal feeding to all volumes of the tank. Under these conditions, the greatest amount of work can be accomplished in a given aeration volume.

2. The BOD level in the aeration basin is essentially equal to the effluent BOD. Therefore, the biological culture is not subjected to strong wastes for a significant period of time.

3. Lysis in the biological culture releases nutrient materials. In the homogeneously-mixed system, this release and the demand for nutrients occur at the same point. Therefore, if addition of nutrients is necessary, the possibility for reuse in the homogeneously-mixed system will minimize these additions.

As a result of evaluations of various plans for homogeneously-mixed basins, both separate from and in a common basin with the final clarifier, considerations of various mixing facilities to provide an oxygen transfer rate in excess of that obtained by bubble aeration alone, and cost comparisons, it was decided to use activated sludge units which provide aeration and final clarification in a single circular unit. An oxygen transfer efficiency of 25 per cent will be provided by the mixers. The capacity of the activated sludge units will be equivalent to that which would have been provided in separate units for aeration and settling.

Plant Description

General

The domestic sewage will pass through comminutors, a Parshall flume, a grit removal basin, parallel primary clarifiers, and a chlorine contact basin. Upon leaving the chlorine contact basin, one-third of the flow will pass directly to the activated sludge system. The industrial waste will pass through a Parshall flume, grit removal basins, and parallel primary clarifiers. Upon leaving the primary clarifiers, the flow will be split and one-third will pass through the pH adjustment facilities, nutrients will be added, and the flow will continue to the secondary process. Sludge from the domestic primary basins will be thickened with waste activated sludge prior to vacuum filtration.

Industrial Clarifiers

Peripheral feed circular clarifiers were selected for this service. The flow enters one side of the basin, flows around the periphery and under a skirt from which the flow is upward to mul-

tiple collection troughs. This type of basin was selected for two reasons: (*a*) the relative ease in protecting the steel sludge collector mechanisms with coatings as opposed to the difficulty of protecting chain and sprocket drive collector mechanisms for rectangular basins, and (*b*) the fact that the industrial waste will contain solvents and other floating inflammables and it is considered desirable to contain the inflammables at the perimeter of the tank where fire may be more easily extinguished.

The two clarifiers are designed to provide a detention of one hour at average flow. They will be 64 ft in diameter and have a sidewater depth of 11.5 ft. Sludge will be removed continuously and discharged to the plant effluent until disposal facilities are later constructed.

pH Control Facilities

As has been previously mentioned, the pH of the waste stream may vary between the extremes of 1.0 to 13.0, with a change rate as fast as one pH unit in 10 seconds. This pH variation coupled with a possible stream flow rate change from 5,600 to 11,000 gpm over a 5-min period, presents some rather difficult problems. These problems were studied in the Company and the system design was simulated on an analog computer. The design became the subject of a publication by Field (1).

The neutralization process is carried out in series with the following equipment sequence: influent to Blender A to first-stage mixer and control to Blender B to second-stage mixer and control to Blender C to effluent.

The pH control is accomplished by additions of commercial grade sulfuric acid or 50-per cent caustic solution to the stream in accordance with the load demand. The first- and second-stage mixers each have a separate feedback control loop. Except for reagent valve sizes, the two closed loops require identical equipment and exhibit the same dynamic behavior.

When the stream concentration assumes extreme values (pH 1 to 2 or pH 12 to 13), reagent is added immediately to Blender A by open loop control as pretreatment. The first-stage treatment operates between pH 2 and 12, and the second-stage treatment serves as a final adjustment over the span pH 4 to 10, to give a final pH between 6.0 and 9.0.

The three blenders will each have a volume of at least 7,500 gal and will be agitated by mixers constructed of stainless steel. The first- and second-stage mixers will be of the pipeline type with 300-gal capacity between the mixing zone and the following blender. Construction will be of stainless steel or properly coated and protected steel.

The system is designed for neutralization of the full flow and will be so constructed, except for the sizing of the reagent valves.

Nutrient Addition

The industrial waste will require the addition of nitrogen and phosphorus, which will be supplied in the form of anhydrous ammonia and phosphoric acid. These nutrients will be added to the waste after pH adjustment.

Comminutors and Screens, Domestic Flow

Two comminutors of a size to handle the maximum flow rate in parallel will be installed. A by-pass bar screen will be provided for emergency use.

Domestic Clarifiers

The two domestic clarifiers will be circular and of the same type as the industrial clarifiers. They are designed for two hours of detention at the average flow. The basins are 64 ft in diameter.

Chlorination Facilities

The chlorine contact basin will provide a detention of 15 min at the aver-

age design flow. Chlorinator capacity will be such that each of two machines can provide a dosage of 20 ppm chlorine at average flow. Prechlorination at the domestic sewage pump station will be provided for odor control. The chlorinators will be automatically controlled in proportion to the rate of flow entering the plant.

Activated Sludge Units

Two circular units are being provided for the activated sludge process. The units will be 90 ft in diameter by 24 ft, 6 in. of water depth. The aerator units will be powered by variable speed drives. The mixer in each basin will have capacity to disperse 4,000 cfm of air at an oxygen transfer efficiency of 25 per cent.

The minimum expected air requirement for the activated sludge process is 2,000 cfm, the average requirement 4,000 cfm, and the maximum requirement 8,000 cfm. The air supply will be furnished by blowers in combination to provide steps of approximately 2,000, 4,000, 6,000, and 8,000 cfm.

The BOD load on the activated sludge unit will be 169 lb/day/1,000 cu ft.

The aeration detention time will be 7.67 hr and the detention in the settling zone will be 2.50 hr.

Sludge Disposal

The sludge from the domestic primary basins will be pumped continuously along with waste activated sludge to a sludge thickener. The thickener will be 45 ft in diameter by 10 ft of side water depth. Because of a high load of industrial BOD in the secondary system, the ratio of waste activated sludge solids to primary domestic solids will be higher than usual. The thickener is expected to concentrate the mixed sludge to a solids concentration of 4 to 6 per cent.

The thickened sludge will be further dried on vacuum filters. Two units will be installed and the filters will be designed to operate one shift per day for five days each week. Sludge will be taken to the City dump until further disposal means are determined for the industrial primary sludge previously mentioned.

General Features

Because of the necessity for providing and maintaining coatings for protection against the industrial wastes, the flow between basins will be carried in open channels. Slide gates constructed of stainless steel will be used in the industrial waste flow and gates constructed of aluminum will be used in the domestic flow. Pneumatic gate operators will be employed for automatic operation at certain points.

Sulfuric acid, 50-per cent sodium hydroxide, phosphoric acid, ammonia, and ferric chloride will be obtained in tank truck quantities and stored in steel tanks at the plant.

The connections to the secondary process will be such that all or any portion of the domestic flow and any desired portion of the industrial flow can be directed to the activated sludge system, so that the full capacity of the secondary process can be utilized.

Preliminary Engineering Cost Estimates

Total Project Cost

Preliminary engineering estimates show the total project cost to be $8,623,000.00, of which Union Carbide Chemicals Company's portion is $6,327,000.00, and the City of South Charleston's portion $2,295,000.00.

Annual Costs

Preliminary engineering estimates show the total annual utilities and reagent cost to be $172,400.00, of which $157,800.00 is Carbide's portion and $14,600.00 is the City's portion.

Based on current interest rates on "AAA" Municipal Revenue Bonds and preliminary estimates of project cost, the total annual debt service cost will be $608,000.00, of which Carbide's portion is $446,000.00 and the City's portion is $162,000.00.

General

The high ratio of first cost to volume of wastes to be treated is attributable to the necessity for use of special alloy metals and expensive protective coatings to withstand corrosiveness of industrial wastes before pH adjustment.

The high cost of utilities and reagents is attributable to reagent cost, Carbide's portion of which is estimated at $87,900.00 per annum.

Reference

1. Field, W. B., "Design of a pH Control System by Analog Simulation." *I.S.A. Jour.*, **6**, 1, 42 (Jan. 1959).

MEETINGS OF INTEREST

Feb. 21–24	**American Institute of Chemical Engineers,** Biltmore Hotel, Atlanta, Ga. F. J. Antwerpen, Executive Secretary, 25 West 45th St., New York 36.
Mar. 1	**Fourth Great Plains Sewage Design Conference,** Castle Hotel, Omaha, Nebr. W. F. Rapp, Jr., Division of Sanitation, State of Nebraska, Lincoln, Nebr.
Mar. 7–11	**American Society of Civil Engineers,** Jung Hotel, New Orleans, La. W. H. Wisely, Executive Secretary, 33 West 39th St., New York 18.
Mar. 16–18	**Twenty-Third Annual Short Course for Superintendents and Operators of Water and Sewerage Systems,** Louisiana State University, Baton Rouge. Dean Fred H. Fenn, College of Engineering, Louisiana State University, Baton Rouge.
Mar. 29–30	**Second Joint Conference on Gaseous and Liquid Waste Problems,** Hotel Traymore, Atlantic City, N. J. Mid-Atlantic States Section of the Air Pollution Control Assn. and Industrial Wastes Assn.
Apr. 4–7	**6th Nuclear Engineering and Science Conference, 8th International Atomic Energy in Industry Conference, 6th International Atomic Exposition,** New York Coliseum, New York. D. P. Reynolds, c/o Engineers Joint Council, 29 West 39th St., New York 18.
Apr. 4–8	**Course on River Pollution, Sewage Disposal, and Trade Waste Treatment,** University of Birmingham, Edgbaston, Birmingham, England. R. F. Wills, Civil Engineering Dept., The University, Edgbaston, Birmingham 15, England.
Apr. 5–14	**137th Meeting of the American Chemical Society,** Hotel Netherland-Hilton, Cleveland, Ohio. American Chemical Society, 1155 16th St., Washington 6, D. C.
Apr. 7–8	**Ninth Annual Municipal and Industrial Wastes Conference,** North Carolina State College, Raleigh. Division of College Extension, P. O. Box 5125, State College Station, Raleigh, N. C.

UTILIZATION OF SPENT SULFITE LIQUOR*

By J. M. Holderby and W. A. Moggio

General Manager, and Production Manager, Lake States Yeast & Chemical Division of St. Regis Paper Company, Rhinelander, Wis.

Basically, the sulfite pulping process involves the separation of the cellulosic constituents of wood from the non-cellulosic by dissolving the latter through complex chemical reactions. On a dry wood basis, the yield of insoluble cellulosic materials is in the neighborhood of 50 per cent, the remainder comprising the dissolved lignosulfonates, sugars, and other minor constituents. Herein lies the problem faced by the sulfite pulping industry, specifically, the disposal of the residual pulping liquor containing the dissolved substances. Disposal must be achieved without creating undesirable conditions in the receiving bodies of water.

The limiting factor in the solution of the spent sulfite liquor disposal problem is one of economics rather than technology. Technically sound processes for the disposal of these effluents have been developed involving purely treatment or utilization techniques. However, when examined in the light of economics these processes have generally been unattractive.

The practical methods of resolving this problem of spent sulfite liquor to date include simple disposal methods and utilization methods. The former methods include disposal by dilution, either directly to the receiving body of water or by the use of storage lagoons and controlled discharge. The other simple disposal method used is land disposal through the mechanics of soil filtration or as roadbinder. Where simple disposal by dilution or land disposal are not practical, the spent sulfite liquor is usually further processed and suitable utilization of the end product(s) is sought. Full scale installations for utilization include (*a*) evaporation and burning for the recovery of the fuel value in the liquor and, in some instances, recovery of spent pulping chemicals, (*b*) evaporation with or without drying for the preparation of lignosulfonate compounds and derivatives which find limited industrial application, or (*c*) fermentation of the spent liquor sugars for the production of yeast or alcohol.

It has long been recognized in the sulfite pulping industry that utilization of the non-fibrous half of the dry pulpwood log will supply the eventual solution of the spent liquor problems. It has become increasingly evident that the traditional or standard effluent treatment processes cannot be economically applied. A review of the technical and patent literature on spent sulfite liquor will confirm this reasoning. Since the sulfite pulping process was first commercialized some 80-odd years ago, there have been literally scores of schemes worked out for liquor utilization and hundreds of products have been prepared from this intriguing material. The difficulty has been, with relatively few exceptions, that the products either were not quite good enough or were too expensive to meet competition from similar products produced from other sources, or that markets for the products were too limited to make production economically feasible.

Rhinelander Paper Company, the

* Presented at the 14th Industrial Waste Conf., Purdue University; Lafayette, Ind.; May 5–7, 1959.

TABLE I.—Composition of Spent Sulfite Liquors from Spruce Wood

Component	Concentration* (g/l)
Neutralized solids	100
Total sugar (as glucose)	15–22
Hexose	11–16
Pentose	4–6
Volatile acids (as acetic)	2–5
Sulfur (all forms as SO_2)	8–10
Total inorganic (free SO_2)	0.5–2.5
Organic (loosely combined SO_2)	3.0–5.0
Lignin (as lignosulfonate)	50–65
Calcium	7–10
Miscellaneous compounds	2–5
pH	1.5–3.0

* Variations based on liquors of the same solids concentration.

largest glassine and greaseproof paper manufacturer in the United States, employs utilization processes for handling its spent sulfite liquor. The spent liquor comes from the daily production of approximately 110 tons of sulfite pulp produced by the calcium bisulfite process from both hardwoods and softwoods.

The techniques used for processing of the spent liquor encompass the production of nutritional yeast from the sugars, followed by evaporation, with or without drying, of the remaining lignosulfonates. These by-products, yeast and calcium lignosulfonates, are sold in appropriate markets. Facilities for the burning of the evaporated liquor for its fuel value are also available in the event of insufficient sales outlets for the calcium lignosulfonates. The production of nutritional yeasts and calcium lignosulfonates is the function of the Lake States Yeast and Chemical Division, which like the Rhinelander Paper Company that it services, is a division of St. Regis Paper Company. These products are also marketed under Lake States' trademark as Dried Torula Yeast and Toranil A and Toranil B as the liquid and dry forms, respectively, of calcium lignosulfonates. This plant is the pioneer producer of Torula Yeast in the United States and one of the earliest producers of desugared calcium lignosulfonates.

Spent Sulfite Liquor

The liquor results from the treatment of wood chips under conditions of elevated temperature and pressure with an aqueous sulfurous acid solution of calcium bisulfite. During the digestion period lignin combines with the sulfur dioxide or sulfite ion to form calcium lignosulfonate. After the cooking process is completed the acid liquor containing the dissolved materials is separated from the insoluble cellulose. Other bases such as sodium, ammonium, or magnesium can be used in place of calcium to form the cooking acid.

Approximately one ton of dissolved wood solids is produced with each ton of pulp made. The organic matter present in the liquor is complex, with only about 20 per cent of the dissolved solids being made up of easily oxidized sugars and related compounds. Thus, some 400 lb of fermentable material is produced for each ton of pulp made, or 20 tons per day for a pulp mill producing 100 tons of pulp per day. Of this quantity, only 14 to 16 tons of sugars are usually collectable at a practical concentration.

The concentration of the spent liquor from different mills ranges from 60 to 150 g of solids per liter (500 to 1,250 lb/1,000 gal). The variation results from the type of wood used and the degree of cooking required to produce the desired pulp quality. Liquors from the pulping of hardwoods have a higher proportion of sugars (up to 30 per cent on a solids basis) than softwood liquors (15 to 22 per cent on a solids basis). The types of sugars are also a function of the wood used. Softwoods yield a product with a total sugar of about 75-per cent hexoses,

which are largely mannose. Liquors from hardwoods may have as much as 70-per cent pentose sugars, mostly in the form of xylose. Hardwood liquors may also have a higher content of acetic acid. Table I (1) shows the approximate composition of spent sulfite liquors from spruce wood.

Total volume of spent liquid produced will range between 2,000 and 3,000 gal per ton of pulp, and the total solids in the spent liquor from one ton of pulp will range from 2,000 to 3,000 lb. These figures are variable, depending on the duration and severity of the pulping cycle, the cooking liquor concentration, fiber yield from the wood, and the type of wood being pulped.

Economic considerations in any processing operation usually require that the material to be processed should be in as concentrated form as practicable. In the case of spent sulfite liquor, the efficiency of the spent liquor collection system is of great significance. Collection system efficiencies probably vary from less than 50 to about 90 per cent based on the total solids produced by the pulping operation. The efficiency will vary depending on the equipment used for draining and displacing the spent liquor from the pulp.

As would be expected, the BOD of spent sulfite liquor is also variable. It is largely controlled by the sugar content, although other dissolved substances contribute substantially. Generally, the BOD range of spent sulfite liquor at this plant is from 25,000 to 35,000 mg/l.

Yeast Production

History

The idea of growing microorganisms on the sugars present in spent sulfite liquor is not new. Ethyl alcohol has been produced through the culture of *Saccharomyces* yeasts in this substrate for a great many years. Probably the first thought of converting wood sugars into edible protein was in Germany. The idea was seemingly born during World War I when it became necessary to seek out new protein sources to circumvent the Allied blockade. The process was first put into large scale commercial production during World War II. Common yeast (*Saccharomyces*) might have been used for the process but wood sugar is a mixture of both five- and six-carbon sugars and this variety of yeast can assimilate only the six-carbon sugars. A number of different yeast species were actually used in Germany but one of them, then classed as *Torulopsis utilis* and since reclassified as *Candida utilis*, appeared to be most efficient.

The Sulphite Pulp Manufacturers' Research League in this country started growing yeasts on spent liquor in 1943. After screening many organisms for growth characteristics and dried cell composition, they too chose to concentrate on *C. utilis*. When hostilities ended they obtained detailed information on the German developments, fitted it in with their own pilot plant experience and proceeded to lay out a commercial, continuous flow process, which eventually became the Lake States organization.

Factors for Growth

Since the major BOD-producing constituents of spent sulfite liquor are the sugars—five-carbon and six-carbon—*Candida utilis*, or torula yeast, became the organism of choice because of its ability to assimilate both types of sugars. Torula yeast cultures are relatively easy to maintain and are routinely propagated with little evidence of instability or variation. One of the remarkable features of torula yeast has been the ease with which large fermentation plants may be inoculated and maintained in continuous operation with apparent freedom from serious contamination problems. Torula yeast grows rapidly and is able

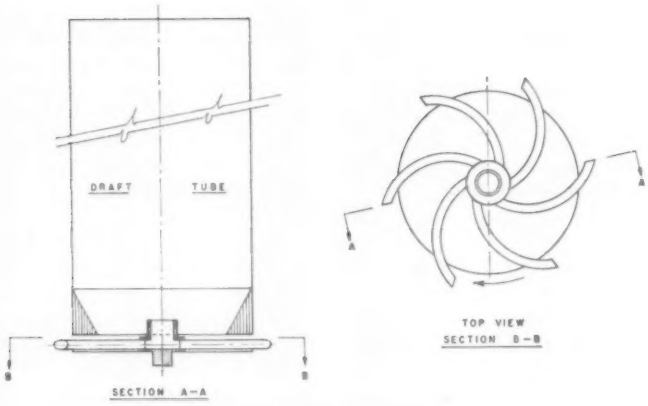

FIGURE 1.—Waldhof aerator wheel.

to synthesize all growth requirements from simple compounds. Bacterial contamination is abnormal with properly controlled, continuously fed torula yeast growth.

Factors of prime importance for reduction of BOD of spent sulfite liquor through vegetative growth of torula yeast are the concentration and rate of feed of liquor, the air supply, inorganic nutrients, temperature, and pH control.

Liquor preparation involves reducing the sulfur dioxide of the process feed to a point where it will not inhibit yeast growth. Sulfur is present in the raw liquor in a number of forms —which for convenience may be termed (*a*) "inorganic" or that which will reduce iodine in a cold acid solution, (*b*) "organic loosely combined" which reduces iodine in acid solution only after first being split from its organic complex by strong alkali treatment, and finally (*c*) "firmly combined sulfur" which does not reduce iodine even after alkali treatment. This firmly combined sulfur does not interfere with yeast growth while the other two forms do.

Growing Conditions

Effective liquor preparation can be accomplished by lime treatment, which in general results in the precipitation of calcium sulfite, or by conventional stripping in a tower with steam or other gas.

In common with other continuous processes involving a time reaction, it is essential to provide sufficient residence time in the reactor (in this case the fermentor) to permit the desired reaction to attain virtual completion. The time it takes a single yeast cell to bud and split into two mother cells will vary widely, depending on nutritional and environmental conditions. Experience in growing torula yeast indicates a practical minimum holding time of approximately three and one-half hours.

The rate of addition of the fermentables must be adjusted to parallel closely the rate of usage by the yeast culture. It appears that yeast growth is most vigorous when the concentration of fermentables in the wort is held at a minimum. In effect, it appears desirable to approach the point of cell starvation but never to permit actual shortages to occur.

The growing of yeast, being an aerobic process, requires oxygen which is normally furnished as air. The quantity of air supplied to the fermentor and the manner of its distribution are critical factors in yeast growth

and the assimilation of the liquor sugars. Due to the foaming characteristics of spent sulfite liquor, a mechanical aeration system—the so-called "Waldhof" unit shown in Figure 1—which incorporates a foam control feature is ordinarily employed (2)(3). High efficiencies of oxygen transfer at the air-liquor-yeast interface are obtained through the formation of emulsions having as much as two volumes of air for each volume of liquor. The amount of air required for efficient growth of yeast is critical. Too much air can be accompanied by increased carbon dioxide respiration, increased heat formation, and lowered yeast yield. Too little air results in anaerobic fermentation conditions favoring alcohol production and lowered yeast cell yields. Air requirements are difficult to specify because the efficiency of aeration (oxygen transfer) varies with different substrates and with types and sizes of equipment used for aeration. Satisfactory aeration with spent sulfite liquor substrates have been obtained with as low as 150 cfm of air per pound of dry yeast produced.

Nutrients and Temperature

For efficient growth, yeast requires nitrogen, phosphorus, and potassium in relatively large amounts; and calcium, magnesium, copper, iron, and certain other elements in smaller amounts (4)(5). Spent sulfite liquor is almost totally lacking in nitrogen, phosphorus and potassium, and in some instances magnesium. The source of these inorganic elements furnished to the yeast depends on the cost, availability, and effect on the growth of the yeast.

The growth of yeast under aerobic conditions results in the liberation of large amounts of heat, necessitating the use of cooling water or refrigeration. The quantity of heat released per pound of yeast will vary with different substrates. Fermentation of pentose sugars results in the release of larger quantities of heat than the hexose sugars. In the case of spent sulfite liquor from pulping spruce wood, approximately 1,700 cal of heat are evolved per pound of dry yeast produced (1). Efficient yeast growth on a continuous basis requires the maintenance of an even temperature in the fermentor. For normal growth of torula yeast, the temperature is maintained in the range of 93 to 98°F (4) (5). Increased temperature results in reduced efficiencies and several days may be required for the yeast to recover from heat shocks.

pH

Torula yeast will grow satisfactorily in a pH range of 4.5 to 6.0 (5). Control of the substrate pH, however, is important for a number of reasons. Where higher temperature operation is required and where sugars are fed in high concentrations, the lower pH levels are used. This is done to prevent bacterial contamination, which is more likely to occur at high pH and temperature levels, particularly when the holding time in the fermentor is long because of the high sugar concentration feed. Control of pH is conveniently accomplished by the automatic or manual addition of ammonium hydroxide, although other common basic substances may be used.

Harvesting

Recovery of the cell material produced in the fermentor is strictly a matter of physical separation. Filtration, utilizing conventional vacuum filters, has been used to some extent but usually has been of dubious merit due to the tendency of the yeast to blind the filter cloths. Solid bowl centrifuges have been used successfully but have the disadvantage of batch operation. Most torula yeast produced today is recovered in nozzle type continuous centrifuges which are operated

TABLE II.—Typical Analysis of Dried Torula Yeast

Component	Per Cent
Moisture	6.5
Crude protein (N × 6.25)	52.0
Crude fat (with prehydrolysis)	4.5
Minerals (ash)	7.8
Carbohydrates (by difference)	29.2

in series, with dilution between stages, to accomplish the desired degree of washing.

Drying is an essential step in the production of nutritional yeast. Heat is used to inactivate the cells as well as to dry the yeast to a powder form. Spray drying has been used to some extent in Germany for the latter step but in this country drum drying is practiced exclusively.

Depending on the total content of fermentable materials in the liquor, a yield of dry yeast substance approaching 50 per cent by weight may be expected. A typical analysis of torula yeast produced from sulfite liquor is shown in Table II.

Yeast Plant Facilities

Preliminary Procedures

Figure 2 shows a schematic flow diagram of the yeast plant which is located adjacent to the paper mill.

Spent sulfite liquor from the pulp blowpits is pumped through a rotary screen to remove pulp fibers from the liquor and then passes into two storage tanks, each of 125,000-gal capacity. These tanks are fabricated of fir wood staves and all the piping, pumps, and screens are of stainless steel construction. Besides serving as storage, the tanks also serve as blend tanks for leveling out sugar concentrations in the raw liquor. Spent liquor comes from the pulp mill intermittently, depending on the wood cooking cycle, and the total quantity collected is approximately 125,000 gpd. Fermentable sugar content of the liquor ranges from 2.0 to 3.0 per cent and dissolved solids range from 10 to 12 per cent. Temperatures of the liquor is about 175°F and pH is about 2.5.

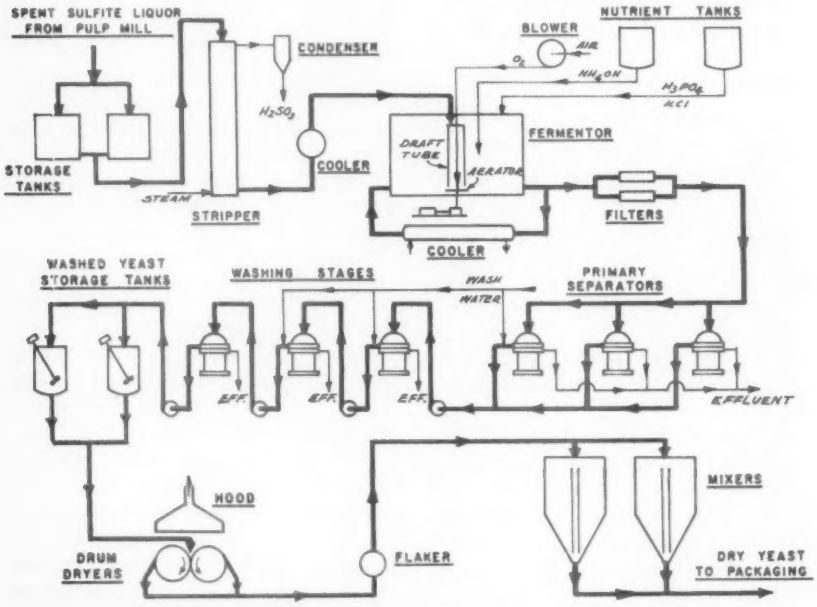

FIGURE 2.—Schematic flow diagram of yeast production plant.

Before the spent liquor can be fed to the fermentor it is necessary to reduce the sulfur dioxide content to a level which does not interfere with yeast growth. This is accomplished in a stripping tower of stainless steel construction packed with ceramic Raschig rings. Feed liquor enters the top of the tower while live steam enters the bottom. Water vapor and sulfur dioxide stripped from the liquor enter a surface condenser and the condensate, a sulfurous acid solution, is returned to the pulp mill for reuse.

The stripped liquor leaves the bottom of the stripper and passes through a cooler, entering at a temperature of about 180°F and leaving at about 80°F. Following another screening step, the cooled stripped liquor enters the fermentor.

Fermenting Operation

The fermentor is a stainless steel clad, carbon steel tank with an open top, 26 ft in diameter and 14 ft high. The tank is provided with a vertical draft tube, 4 ft in diameter, located in the center of the tank. The operating volume of the fermentor is 45,000 gal of emulsion (liquor plus entrained air) or approximately 20,000 gal of liquor (air free). Retention time in the fermentor is approximately 3½ hr at a liquor flow rate of 100 gpm.

The aerator device, located at the bottom of the draft tube, resembles a centrifugal pump impeller, with the open-end air tubes acting as vanes. It is driven by a 250-hp motor. Air is supplied to the aerator wheel by means of a centrifugal blower of 2,000 cfm capacity. Air is conveyed to the aerator wheel by an 8-in. pipe descending through the center of the draft tube, connected through a rotating stuffing box. Air dispersion results from the peripheral shearing action between the end of the air tubes and the liquor. The aerator serves not only as a means of supplying air to the yeast cells in the liquor substrate but also as a circulating pump for the emulsion. The combination of this type of aerator with a draft tube eliminates the need for chemical defoamers to control foaming.

Nutrients required for yeast growth are supplied to the fermentor by means of two solution streams. Nitrogen, in the form of a 20-per cent aqueous ammonia solution prepared from anhydrous ammonia, is fed automatically. Besides furnishing nitrogen in an available form this solution is also used to control fermentor pH. Phosphorus, potassium, and additional nitrogen, all in available form, is fed as a solution of dibasic ammonium phosphate and potassium chloride. Solution feed rate is based on the sugar content of the feed liquor.

Fermentor temperature is controlled in the desired range by circulating the growth medium containing the yeast cells through an external tube and shell heat exchanger. Additional cooling during warm seasons is obtained by spraying cooling water around the outside of the fermentor walls.

The stripped and cooled liquor is added to the fermentor directly into the draft tube. The liquor is sterile and aseptic techniques are not required in order to maintain a pure culture of torula yeast growing in the fermentor. The fermentor feeds of stripped liquor and nutrient solutions are on a continuous basis, the fermentation being a continuous process after the initial inoculation with a culture of torula yeast.

Separation

Assimilation of the sugars by the yeast is followed by a yeast cell separation process to remove yeast from the substrate. Continuous nozzle type stainless steel centrifuges are used for this. The yeast suspension is removed from the fermentor at the same rate as the stripped liquor feed to the fermentor. This suspension, containing 1.0- to 1.5-per cent dry yeast solids, is fed

to the primary centrifuges from a constant level headbox. Two streams are discharged from the centrifugal separators, the yeast cream containing the concentrated yeast solids and the effluent which contains the substrate, freed of yeast cells and relatively sugar-free. The effluent is collected and sent to the evaporator plant for further processing.

The concentrated yeast suspension or yeast cream is collected, diluted with water, and pumped to additional continuous centrifuges used as washing stages. Depending on the final yeast product desired, the yeast cream receives one or more washes, each stage consisting of a continuous centrifuge. Following the last washing stage the yeast cream is pumped, undiluted, to agitated stainless steel tanks. The dried yeast solids concentration in the last stage yeast cream varies from 12 to 15 per cent.

The final washed yeast cream is pumped from the storage tanks to two drum rotary dryers for drying. The drums are cast iron, 3½ ft by 10 ft, and are steam heated at 90 psi. The dried yeast is removed from the drums by a doctor blade, is screened and passes into a hopper from which it is packaged in multiwall bags, fiberboard, or steel drums. The final product is nonviable and contains 6 to 8 per cent moisture.

Calcium Lignosulfonate Production

The search for useful lignin compounds derived from spent sulfite liquor has been going on for many years. One of the major problems has been to obtain these lignin materials in concentrated or dry form at a cost sufficiently low to allow marketing at competitive prices with materials which they are meant to replace, or in areas where only these lignin compounds possess the required characteristics. In both instances, large volume outlets are a prerequisite.

The major technological stumbling block, until recent years, had been the development of a satisfactory solids concentration technique. Evaporation techniques, which offered the most attractive approach, were initially unsatisfactory because of the calcium sulfate scaling problems of evaporator tubes and surfaces inherent in the evaporation of calcium base spent sulfite liquors. Initially, conventional evaporators were used but these had to be shut down at frequent intervals to remove the calcium sulfate scale which formed. This was a very inefficient and costly procedure.

The calcium sulfate scale problem encountered in evaporating calcium base spent sulfite liquors was solved in Sweden during World War II. Several processes became available at that time, the most successful being the development of the Rosenblad channel switching evaporators. The responsible factors for this technological breakthrough in Sweden stemmed from two sources, both of which were important because of the Swedish economic structure but of practically no importance in the United States. The high cost of fuel in Sweden behooved their sulfite pulp industry to look toward recovering the heat value of the organic constituents of their spent sulfite liquor. Further, because of the shortage of industrial alcohol the sugar content of the spent sulfite liquors became a very attractive source of alcohol, in fact, the major domestic source in Sweden.

The development of this type of evaporation unit created the stimulus to evaporate desugared spent sulfite liquor, primarily for the preparation of calcium lignosulfonates for industrial applications. The installation in 1952 of a Rosenblad channel switching evaporator * system marked the first such installation in the United States.

Two types of calcium lignosulfonates are manufactured from the desugared

* General American Transportation Corporation, Chicago, Ill.

spent sulfite liquor, designated as Toranil A and Toranil B. The former is a liquid concentrate containing 50-per cent solids while the latter is a dry powder containing approximately 6-per cent moisture. Toranil A is a coffee colored, viscous liquid with a characteristic tart odor, a specific gravity of 1.24 with a pH of approximately 4.5. Toranil B is a free-flowing, light tan powder with a pH of 4.5 in solution. It is water soluble in all proportions and virtually non-hygroscopic. On a dry solids basis, the Toranil products are 96-per cent calcium lignosulfonate and 4-per cent total carbohydrates, including 1.2 per cent as reducing sugars (glucose). Based on an ash content of 17.3 per cent, the ash constituents are shown in Table III.

Toranil has been employed as an organic reducing agent. On oxidation, it yields various aldehydes, phenols, acids, and cyclic alcohols. Toranil exhibits wetting and dispersing action on a large variety of materials. The number and variety of hydroxyl groups are believed responsible for the fixation of collagen observed in certain types of tanning reactions. Its present uses include adhesives, binding agents, dispersants, core binding, refractory materials, resin extenders, boiler water treating compounds, oil well drilling muds, cement additives, dye mordants, emulsifiers, foaming agents, foam stabilizers, sizing agents, and as a chemical raw material.

TABLE III.—Constituents of Ash in Toranil Products (Total Ash, 17.3 per cent)

Constituent	Per Cent
Calcium and magnesium as oxide	7.50
Sulfate (as SO_4)	8.70
Silica (as SiO_2)	0.60
Iron and aluminum (as oxide)	0.27
Iron and aluminum (as phosphate)	0.03
Insolubles	0.17
Trace materials (Cl, Na_2), (K_2O)	0.03

Evaporation Plant Facilities

Description

A schematic flow diagram of the evaporation plant facilities is shown in Figure 3. The evaporator installation is a four-body triple effect evaporator of a flat plate heating surface type embodying the Rosenblad channel switching system. Construction materials are entirely of Type 361 stainless steel. Total heating surface is 4,650 sq ft, equally divided among the three effects. Steam supply to the evaporator is 125 psi and operating pressures range from 25 psi to a vacuum of 26 in. of mercury. The torula yeast plant effluent, which is the feed source to the evaporators, comes to the unit at approximately 90°F. Essentially, the evaporator comprises a series of vapor bodies with separating chambers of ample proportions to permit the boiling spent liquor to release its vapors, plus heating elements utilizing flat plates for heat transfer surfaces and recirculation lines extending from the lowest point in the vapor body to the bottom of the heating element to permit adequate recycle of liquor for wetted surface evaporation.

The fundamental design change for this type evaporator is in the form of the heating surface, comprising flat plates rather than tubes. These plates consist of long rectangular sheets placed face to face, separated from one another by suitable spacers and sealed one from the other so as to form a channel between each pair of plates. The number of plates so arranged depends on the amount of heating surface required for the evaporation duty. This assembly of plates is enclosed by a reinforcing shell to provide adequate strength against any pressure conditions imposed on the equipment. In all cases, an odd number of plates is

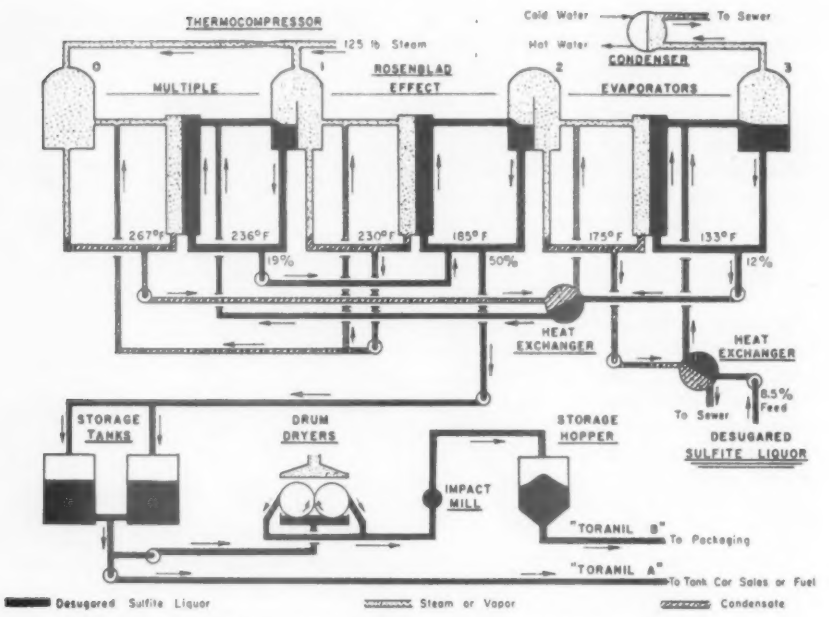

FIGURE 3.—Schematic flow diagram of evaporation plant facilities.

employed thereby creating an even number of channels. By suitable partitions at the ends of the channels, every other channel receives vapor or steam as the heating medium and through the alternate channels the liquor being evaporated is circulated.

Operation

Operation of the evaporator proceeds in the usual fashion. Steam is introduced at the first effect and condensed vapors from the last effect produce desired vacuum while liquor is transferred in a manner best suited to its characteristics. As evaporation proceeds, scale gradually forms on the heating surfaces. At the end of a specified time, six hours in this instance, the liquor side of the units is switched with the condensing steam side, by means of reversing valves at each end of the multiple effect units. This is accomplished by a quick shutdown during which liquor is transferred to what has been the steam side. Upon starting up again, it follows that all portions of the equipment previously subjected to scaling due to the presence of boiling liquor are now being washed by condensing vapors and condensate which is recycled before withdrawal and mixed with vapors entering each heating element.

The net effect of this switching cycle is to create a washing of all surfaces which in the prior operation had become fouled. Further, the washing proceeds simultaneously with normal evaporation, resulting in continuous cleaning of heating surfaces. Scale is actually removed by a dissolving action, the switching interval being set by a very safe relationship developed from rate of scale formation and rate of scale removal by washing. The channel switching operation consumes a total time of 45 min/day. Depending on the characteristics of the liquor being evaporated, scale buildup of as much as one-eighth inch in some parts of the evaporator has been experienced

TABLE IV.—Operating Results

Item	Rate (lb/hr)	Operating Conditions
Feed	39,400	8.5-per cent solids, 90°F
Product	6,700	50 -per cent solids, 185°F
Evaporation	32,700	—
Steam	11,580	125 psi
Economy	2.82*	—

* lb water per pound of steam.

in a six-hour operating period. The scale is a typical white calcium sulfate incrustation.

Based on the production of approximately 80,000 lb/day of calcium lignosulfonate solids from a feed source at 8.5-per cent solids concentration, the following average operating results are shown in Table IV.

The spent liquor at 50-per cent solids concentration and 185°F is discharged from the evaporator into two storage tanks. From these storage tanks the evaporated liquor can be loaded directly into tank cars (Toranil A) or it can be fed to two steam heated (90 psi) double drum dryers for drying to a powder (Toranil B). The dried powder is then processed through an impact mill, into a storage hopper from where it is packaged in multiwall paper bags for shipment. Facilities are also available for burning the evaporated liquor in a furnace for recovery of the heat value of the organic constituents.

Performance

Operating results for the torula yeast plant at Rhinelander Paper Company show a 60-per cent reduction in BOD of the spent sulfite liquor treated. However, reduction in the sugar content of the liquor approaches 90 per cent. The lower figure for BOD reduction is due to the fact that there are significant quantities of BOD producing substances in the liquor which are not assimilated by the yeast cells. It is probable also that some compounds are only partially assimilated, leaving residues which exert a BOD in the final effluent. In addition to the BOD and sugar reductions, there is a 15-to 20-per cent reduction in total solids of the spent sulfite liquor.

Based on a fermentor (aeration tank) capacity of 6,000 cu ft (45,000 gal) and a total loading of 32,000 lb BOD per day (average liquor BOD is 0.262 lb/gal and average flow rate is 85 gpm), the routine loading of the yeast production process is 5.43 lb BOD per cubic foot of fermentor capacity per day. Based on the 60-per cent BOD reduction efficiency, the removal is 3.26 lb BOD per cubic foot of capacity per day. Dry yeast cell yield (daily production of yeast is 12,000 lb) per pound of BOD loaded is 0.375 lb and per pound of BOD removed, 0.63 lb.

Since the yeast plant effluent is given supplementary processing in the evaporation plant, BOD reduction of the spent sulfite liquor processed is virtually 100 per cent. This is likewise true for the sugar and total solids content of the liquor.

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WASTES FROM THE PROCESSING OF CORN CHIPS*

By Ralph Porges

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There is need for an understanding of industrial processes, water demands, and waste discharges of some of the lesser known types of manufactories. This need may grow as the limits of water resources are reached and as the practice of waste treatment is increased for the protection of downstream water users. One of these smaller industries is the production of corn chips.

Plant Process

The paucity of information on corn chip processing in the literature is understandable considering the recent development and promotion of the product. A Mexican-type food, the corn chip had its start in the early 1930's and has since blossomed to a national product.

The basic ingredients are white and yellow corn. The desirable moisture content of the corn ranges from 12½ to 14 per cent.

Whole kernel corn, added to a solution of lime water, is boiled in kettles. It is allowed to soak, during which time the moisture content of the corn increases. The soak water is discharged and the corn is washed to remove husks and tips; some liquor is also removed. The washed corn is ground and pressed to form a dough. Some liquid waste is discharged in the grinding process. The dough is then extruded in chip form to deep fat cookers and, after cooking, the finished chips are packaged. The final product contains one per cent moisture and about 36 per cent oil. Figure 1 presents a flow diagram of the process indicating water use and sources of wastes.

Method of Study

Wastes from a corn chip plant were sampled on a 24-hr basis for five consecutive days. Analyses included BOD and suspended solids; water use was recorded. Data on materials and production, secured from the corn chip plant, included weight of corn processed and solids content, weight of chips produced, oil and moisture content of the finished chips, and processing details. Results were compiled on the basis of a production unit of 1,000 lb of corn processed. A determination was made of the 5-day BOD of whole corn, permitting correlation of the observed BOD in the sewer with that estimated from the loss of corn solids.

To ascertain the BOD of whole corn, a known weight of corn was ground in a mortar and blended to a measured volume of mixture. A portion of the blended mixture was used for a BOD determination according to "Standard Methods." The dry solids content of the corn was also determined. The pounds BOD per pound dry solids could then be calculated.

Results

Unit values relating to plant operation are presented in Table I.

Each 1,000 lb of corn yields approximately 756 lb of corn solids. The population equivalent for each 1,000 lb of

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corn processed, based on BOD,* is about 380 and, based on suspended solids, about 370.*

BOD from Solids Balance

Processing capacity may be expected to vary from plant to plant. However, some representative values will indicate the scope of the waste problem. A plant processing 2,000 lb/day of corn may discharge 5,600 gpd of waste with about 126 lb BOD or a waste equivalent to a population of 760, and 150 lb of suspended solids, equivalent to a population of 750. A 10,000-lb/day plant will discharge 28,000 gal having a BOD of 630 lb (3,780 population equivalent) and 750 lb of suspended solids (3,750 population equivalent).

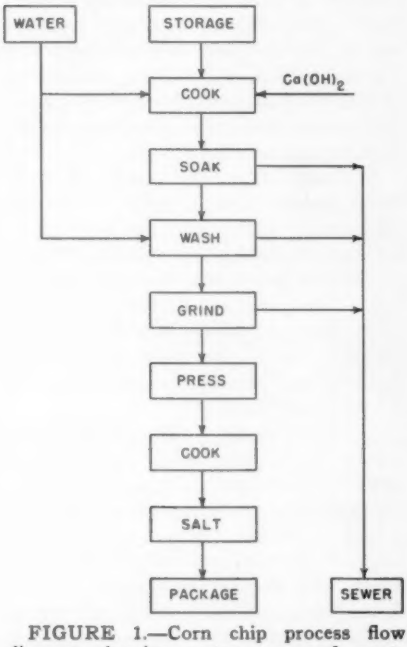

FIGURE 1.—Corn chip process flow diagram showing water uses and waste sources.

A study of potato chipping wastes (1) indicates that the strength of wastes discharged can be approximated from an evaluation of the process and a determination of the solids reaching the sewer. Accordingly, the corn chip process was also evaluated on a dry solids basis and the estimated waste, based on solids discharged, was compared with the waste collected from the sewer.

Results of this evaluation are presented in Table II.

TABLE I.—Wastes Discharged per 1,000 lb Corn Processed

Waste Flow (gal)	5-day BOD		Suspended Solids	
	mg/l	lb	mg/l	lb
2,800	2,700	63.1	3,220	74.8

A separate analysis of whole corn indicated (*a*) 5-day BOD of 473,000 mg/l, and (*b*) solids content of 82.25 per cent. Therefore, for each pound of dry corn solids a 5-day BOD of 0.575 lb will be exerted. This value may be compared to 0.453 lb BOD for a pound of dry potato solids (1). Based on the pounds of dry corn solids lost (Column 3), the amount of BOD discharged is 65.6 lb (Column 4). Column 5 gives the amount of BOD obtained from the sewage analyses — 63.1 lb. The closeness of these results indicates the validity of using the solids balance for estimating strength of wastes in the absence of laboratory data.

This method may be formulated as follows:

$$X = B(mC - D) \quad \ldots\ldots (1)$$

where

X = Pounds 5-day BOD discharged in the waste per day,

C = Pounds of corn processed per day,

m = Per cent dry weight of corn,

D = Dry corn solids produced per day, and

B = Pounds 5-day BOD per pound dry solids of corn = 0.575.

* Population equivalent of 5-day BOD = 0.167 lb; population equivalent of suspended solids = 0.20 lb.

TABLE II.—Solids Balance and BOD Relationship per 1,000 lb Corn

Dry Weight* (lb) 1.	Dry Corn Solids Produced (lb) 2.	Dry Solids Lost (lb) 3.	BOD† of Dry Solids Lost (lb) 4.	BOD in Sewer (lb) 5.
870	756	114	65.6	63.1

* Moisture content = 13 per cent.

† Pounds 5-day BOD per pound dry solids = 0.575.

For example, it may be desirable to evaluate wastes from a corn chip plant that processes daily 6,000 lb of corn having an 86 per cent solids content. It will be assumed that the dry solids produced amount to 4,460 lb. The BOD discharged may be estimated by utilizing Equation 1 and the result is 402 lb of 5-day BOD discharged in the waste per day. This amount of BOD would be equivalent to a population of 2,400.

Discussion

It is recognized that the study of a single plant may not develop factors applicable to other installations. However, calculation of wastes on a dry solids basis appears reasonable and should prove of general value. This was shown in a more extensive study of the potato chip wastes (1). Sanitary wastes and miscellaneous housekeeping may influence the amount and strength of waste from a plant although it is probable that these influences are minor.

There is a lack of published data relative to corn chipping processing and wastes. A recent article (2) on the preparation of tortillas describes home processing of this food in Mexico. The handling of the corn to obtain tortilla dough appears similar to that employed in preparation of the dough for corn chips. Tortilla preparation involves the addition of one part whole corn to two parts of a one-per cent lime solution. The mixture is heated to 80°C for 20 to 45 min and allowed to soak overnight. The liquor is decanted and the corn, now referred to as "nixtamal," is washed two or three times. The nixtamal is then ground to a fine dough called "masa." The dough is cooked on both sides on an iron plate to make tortillas. An experimental preparation (2) of masa from white corn with 15.9 per cent moisture resulted in average total solids lost of 17 per cent. Processing yellow corn with 12.2 per cent moisture resulted in average total solids lost of 14 per cent. This amounts to a solids loss of 143 lb/1,000 lb white corn and 123 lb/1,000 lb yellow corn, compared to 114 lb loss from corn chip manufacturing, a reasonable comparison considering process variations.

A study of wastes associated with the dehydration of hominy (corn) by DeMartini, Moore, and Terhoven (3) indicates the weight in pounds of BOD discharged per 1,000 lb of hominy is 126. This amount is twice that from corn chipping, which is not unreasonable in view of the more complicated chemical and mechanical processing in dehydrating hominy.

In corn chipping, water is used primarily for soaking and washing the corn, washing equipment, and for general cleanup. Good housekeeping may reduce somewhat the volume of water used. A certain amount of waste material will require treatment. Discharge to municipal sewerage when available is probably the best disposal method. Treatment methods that may be considered, where treatment is necessary, are waste stabilization ponds, land application or spray irrigation, and trickling filters. The waste disposal problem should be one of the factors controlling selection of the plant site.

Summary

A study was made of a corn chip plant to observe plant processes and to obtain data on water use and waste parameters based on unit production.

The study involved analysis of data from a waste survey of a corn chip plant and correlation of data with production figures and processing methods.

Wastes originate primarily from the soaking, grinding, and washing operations, and from cleanup. Unit parameters evolved from the study, based on 1,000 lb of corn processed, are (*a*) flow in gallons, 2,800; (*b*) 5-day BOD, 63 lb (population equivalent of 380); and (*c*) suspended solids, 75 lb (population equivalent of 370).

Estimates of plant waste may be obtained without a sampling study if the process is evaluated on a dry solids basis. Excellent correlation was obtained by comparing the pounds of BOD per 1,000 lb of corn processed, calculated from the weight of solids lost to waste—65.6—with the pounds of BOD in the sewer—63.1. The dry solids balance is computed by using the dry solids content of the corn and the chips, and allowing for the oil in the finished chip. The pounds of dry corn solids lost to waste times the factor 0.565 lb BOD per pound of dry corn solids results in the pounds of BOD discharged.

Acknowledgment

Acknowledgment is gratefully accorded the cooperation of Principal Engineer, Arthur D. Caster, and Supervising Industrial Waste Engineer, Fenton H. Dobbs, Sewage Disposal Section, City of Cincinnati.

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BOND SALES IN THE U. S.

The Public Health Service has recently released a publication entitled "Water and Sewage Bond Sales in the United States July 1956–June 1959." This is a 25-page publication, mostly tabular, which gives listings of water and sewage bond sales by the following categories: (*a*) by population size groups, (*b*) by months, (*c*) by issuing authority, (*d*) by type of issue, (*e*) by States, and (*f*) by net interest cost ranges.

The Service has stated that they except to continue this program of compilation and data assembly for use by anyone interested. Inquiries should be directed to:

U. S. Department of Health, Education, and Welfare
Public Health Service
Bureau of State Services
Division of Water Supply and Pollution Control
Basic Data Branch
Washington 25, D. C.

Stream Pollution

COMMERCIAL-SCALE OPERATION OF TURBINE AERATION ON WISCONSIN RIVERS *

By A. J. Wiley and B. F. Lueck; and R. H. Scott and T. F. Wisniewski

Respectively, Sulphite Pulp Manufacturers' Research League, Appleton, Wis.; and Wisconsin Committee on Water Pollution, Madison, Wis.

This paper is the fifth in a series describing cooperative state-industry stream studies conducted jointly by the Wisconsin Committee on Water Pollution, the Sulphite Pulp Manufacturers' Research League, and individual producing units of the Wisconsin pulp and paper industry. This is particularly concerned with results of commercial-scale testing of turbine aeration which mechanically introduces oxygen from atmospheric air to flowing waters which have been deoxygenated to critical low levels of dissolved oxygen in the course of microbiological self-purification. Extensive commercial-scale tests were conducted during the summer of 1958 on three Wisconsin rivers subjected to heavy loadings of oxidizable organic matter derived from the wastes of nature and from man-made sources.

The study of stream conditions has been a joint venture for many years by the cooperating agencies. Early studies were primarily of a qualitative nature. In 1955, an intensive program was jointly organized to develop data of more quantitative nature and in much greater scope and detail than any of the agencies operating individually could undertake. It became important to obtain information as to the conditions and degree of pollution which prevailed in each stream during different periods of the year, and the effect of physical factors including stream flows, water temperatures, and loading rates.

The organization of these studies was described in the first paper (1), and the second paper (2) summarized extensive data obtained from cross-sectioning of the Lower Fox River at nine stations distributed along the 39-mile length of that stream.

With this information as a base, studies conducted during the second year of the program were directed toward determination of the stream purification capacity for several Wisconsin rivers (3). It was then possible to evaluate more completely the working load of these various rivers quantitatively in terms of the amount of oxidizable organic matter entering these streams and its relation to the working capacity of the streams for handling the loading received. It was, for instance, shown that as much as 480 tons of ultimate BOD was present in the Lower Fox River between Stations 6 and 7 daily during one of the survey periods, and that downstream stretches were able to reduce that loading by as much as 325 tons daily at an apparent purification capacity rate of 36,000 lb ultimate BOD at

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flow rates of 1,000 cfs in each mile of flowing water. These rates were determined during warm weather when water temperatures reached 28°C. It was realized, of course, that many factors, including a variety of intangibles, enter into the over-all picture leading to high rates of self-purification within the river waters, but it was possible for stream pollution control authorities to calculate (4) the oxygen deficiencies and to suggest more exactly the corrective measures needed to overcome and reduce the load on these rivers and also to suggest ways in which the oxygen deficiency of the waters might be reduced or alleviated through development of methods for introducing oxygen to the stream by artificial means.

These same cooperating agencies had for some years been studying the practical possibilities for mechanical reaeration of a river through carborondum air diffusion units distributed across the headrace and tailrace of the Pixley Dam on the Flambeau River near Park Falls, Wisconsin (5). Aeration had been conducted each year from 1945 through 1956, during the critical season of low dissolved oxygen at that location. The experimental program had indicated that approximately 10 per cent of the oxygen introduced by that method could be absorbed, but although there was evidence of an improvement in stream conditions, it was apparent that large amounts of air would need to be introduced more efficiently to achieve the desired results.

The fourth paper in this series (6) described preliminary research and development studies of turbine aeration as an improved method for introducing oxygen into river waters during critical periods of low dissolved oxygen. Those preliminary studies have since been expanded (7) to include more extensive commercial-scale tests of turbine aeration which are the subject for this paper. It can now be shown that the use of turbine aeration can be employed feasibly and economically to introduce oxygen at hydroelectric power dams with little extra expense or modification of existing equipment, and with reasonable operating costs. Turbine aeration now appears to be an important development which will be used increasingly wherever dams and suitably designed turbines are available to relieve critical conditions of stream pollution and to increase the capacity of rivers for self-purification.

Review of the Data

Turbine aeration studies during the summer of 1958 were conducted at six sites on three different rivers in Wisconsin. The most detailed data are available from a series of 25 analytical runs made during that summer's operation at the Pixley Dam on the Flambeau River. The data from this series of studies were obtained by methods identical or closely comparable to those used in previous tests over the past 15 years at this location, and to a lesser degree the studies reported for other locations can also be compared with stream study data available from prior years (8) (9) (10).

The Flambeau River at this station has been shown to have long-time average flows exceeding 600 cfs 50 per cent of the time and 150 cfs 95 per cent of the time, and the average flows during this period were 407 cfs. This dam is equipped with three power units, of which the No. 6 Morgan Smith horizontal unit with double runner and a Francis type wheel was used for all 25 runs. The unit produces 450 kw with 21.6-ft hydraulic head at full gate opening. This power dam is owned and operated by the Flambeau Paper Division of the Kansas City Star Company. Tests were conducted jointly by the staff of that mill with field crews from the Wisconsin Committee on Water Pollution

and the Sulphite Pulp Manufacturers' Research League.

On the Wisconsin River, aeration was conducted at two different locations. On the upper reaches of the stream, a few miles below Rhinelander, Wisconsin, tests were conducted at the Hat Rapids Dam owned by the Wisconsin Public Service Corporation in a project sponsored by the Rhinelander Paper Company Division of the St. Regis Paper Company. Average flows on the Wisconsin River at this point are 950 cfs 50 per cent of the time and 449 cfs 95 per cent of the time. The average flow through the turbine during the period of the tests was 338 cfs. All tests were conducted on No. 2 turbine, which is a horizontal type unit with multiple runners and with a production capacity of 320 kw at 20-ft hydraulic head.

Downstream on the Wisconsin River additional studies were conducted during the summer months for the second successive year by the research and engineering staff of Marathon, a Division of American Can Company, at Rothschild, Wisconsin. Four Leffel horizontal turbines were used for aeration at this location. The fifth vertical unit is to be modified for aeration in the near future. Each horizontal unit developed 800 kw at 20-ft operating head. Stream flow averaged 1,762 cfs for the entire period of the summer runs.

Turbine aeration was conducted on three dams on the Lower Fox River. At Kimberly, the Kimberly-Clark Corporation operated an Allis Chalmers vertical turbine with variable pitch propeller for the second summer season of turbine aeration during a 7-wk period. The dam at this location operates with a 9-ft head, and the average stream flow was 1,219 cfs during the 7 weeks of operation. This turbine can pass 1,300 cfs.

Downstream, the City of Kaukauna Electric and Water Utility operates the Rapide Croche Dam midway between the Cities of Kaukauna and Wrightstown. Two series of tests were conducted on turbine aeration at this dam. Preliminary tests early in June 1958 were completed by the representatives of the Water Pollution Committee and the Sulphite Pulp Manufacturers' Research League. Later in the season a four-week sustained operation study was conducted jointly by the Kaukauna utility and representatives of the Thilmany Pulp and Paper Company. An 800-kw Allis Chalmers vertical turbine with variable pitch propeller was used in these studies and operated with an 8-ft head, with an average stream flow of 1,040 cfs.

Still further downstream, tests of turbine aeration were conducted at Little Rapids Dam by the Charmin Paper Products Division of the Procter and Gamble Company. Tests at this location were very preliminary and are as yet somewhat inconclusive in that conditions not previously studied were being evaluated. The principal problem at this dam lies in the fact that the water wheels are set at the level of the tail water and thus no suction exists for drawing air into the turbine. One of the five Leffel vertical type turbines having a hydraulic head of 7 ft was equipped with vents to permit forcing air into the compressor at the relatively low rate of 105 cfm. Exact measurements were not made of river flow through the turbine under study, but it apparently was about 750 and 1,000 cfm during the two periods of study.

Data descriptive of the physical facilities at the various sites for the reaeration studies are summarized in Table I.

Summary of Dissolved Oxygen Data

Essential dissolved oxygen data are summarized in Table II.

All of the tests were conducted with water temperatures running in excess of 20°C, and with dissolved oxygen levels ranging downwards from a top

TABLE I.—Descriptive Data on Power Dams and Turbines

River and Location	Miles from Mouth	Flow (cfs)		Type of Unit	Size (kw)	Head (ft)
		50 Per Cent of Time	95 Per Cent of Time			
Flambeau (Pixley)	86.7	609	150	No. 6 Morgan Smith horizontal double runner, Francis type	600	21.6
Wisconsin (Hat Rapids)	340.0	950	449	No. 2 horizontal multiple runner	320	20.0
Wisconsin (Rothschild)	267.4	2,420	1,380	Leffel horizontal turbine	800	20.0
Lower Fox (Kimberly)	26.0	3,610	1,250	Allis Chalmers, vertical variable pitch propeller	900	9.0
Lower Fox (Rapide Croche)	19.3	3,610	1,250	Allis Chalmers, vertical variable pitch propeller	800	8.0
Lower Fox (Little Rapids)	14.0	3,610	1,250	Leffel vertical turbines	185	7.0

average of 2.46 mg/l for the entire season at Rothschild on the Wisconsin River, to the point of complete depletion of dissolved oxygen on the Lower Fox River at Rapide Croche. It should be noted that preliminary tests made at various locations prior to these studies have shown rather conclusively that the efficiency of aeration may be quite good even at relatively high water temperature levels when the degree of dissolved oxygen saturation is relatively low (40 per cent or less). On the other hand, the efficiency of oxygen absorption drops off rapidly when the level of saturation is higher than 40 per cent. Apparently it will not be found efficient or productive to aerate water at temperatures above 20°C when the DO exceeds 3 mg/l. Gains in dissolved oxygen concentration shown by the tests on the Flambeau River and at Hat Rapids on the Wisconsin River were on the order of

TABLE II.—Summary of Dissolved Oxygen Data for Turbine Aeration

River and Location	Temp. (° C)	No. of Analytical Runs	DO Before Aeration (mg/l)	DO After Aeration (mg/l)	DO Gain (mg/l)	Avg O_2 Absorption Efficiency (per cent)
Flambeau (Pixley)	23.4	20*	0.71	2.22	1.51	25.7
Wisconsin (Hat Rapids)	20	1†	0.58	2.07	1.49	15.2
Wisconsin (Rothschild)	20	Weekly	2.46	3.84	1.38	—
Lower Fox (Kimberly)	20	7 Weeks	0.93	1.28	0.35	22.0
Lower Fox (Rapide Croche)	20	2	0.0	1.35	1.35	24.9
Lower Fox (Rapide Croche)	20	Daily	0.30	1.4	1.1	21.0
Lower Fox (Little Rapids)	22.5	3‡	0.26	0.55	0.30	56.0

* Five additional detailed analytical runs made without aeration for comparison.
† Best of 9 runs to develop optimum aeration conditions.
‡ Average of Runs 5, 6, and 7 from total of 9.

1.5 mg/l. At Rothschild the dissolved oxygen level in the aerated water was raised 1.38 mg/l to an average level of 3.84 mg/l for a substantial proportion of the entire stream flow where 4 of the 5 turbines were operating with aeration for the entire summer. Dissolved oxygen data on the Fox River showed increases of 0.35 mg/l at Kimberly and 1.35 mg/l and 1.1 mg/l at Rapide Croche. An average figure of 25.7 per cent for oxygen absorption efficiency was obtained from the 20 carefully conducted aeration runs at Pixley Dam on the Flambeau River. An additional five runs without aeration were conducted at spaced intervals during the Flambeau studies for comparative purposes. Absorption efficiencies on several individual runs exceeded the average by appreciable amounts, and one of these tests ranged to 34.9 per cent at that location. At Hat Rapids the absorption efficiency was shown to be 15.2 per cent, the Kimberly run on the Lower Fox River, 22 per cent, Rapide Croche, 21 per cent, and as high as 56 per cent was obtained in one of the preliminary runs conducted at Little Rapids on the Fox River. From these data it seems quite probable that further development of equipment design and operation techniques may eventually lead to oxygen absorption efficiencies in the range of 25 to 35 per cent, or even more. The high level of 56 per cent apparent absorption at Little Rapids is probably chiefly valuable in showing the potentials for further development of this aeration technique. That preliminary run utilized much lower quantities of air than were actually justified by prevailing water conditions, and thus contributed to high efficiency. However, 105 cfm represented all the air that could be made immediately available from compressor equipment at hand and in the absence of a suction head which permitted introduction of larger amounts of air under natural draft conditions to the turbines used for the other turbine aeration runs reported.

Summary of Power Data from Turbine Aeration

Table III compares information obtained from the various aeration tests in terms of the power required to introduce a given amount of oxygen. These data confirm the earlier preliminary data derived from tests of 1957 showing that the costs of turbine aeration would probably be economically supportable in terms of the substantial amounts of oxygen introduced to the river waters. For comparative purposes, the total amount of oxygen absorbed was calculated to a common factor based on pounds of oxygen absorbed per 1,000 cfs of stream flow per day. It was felt that there was a particular need for obtaining accurate data in this category in the tests on the Flambeau. At other locations the oxygen absorbed was calculated or estimated as closely as the data would permit. The results were constant and ranged from a low of 1,600 lb oxygen to a high of 8,200 lb oxygen absorbed per 1,000 cfs per day. However, at none of these installations was it felt that there was the ultimate in design and installation of equipment to achieve maximum efficiencies and oxygen absorption, and it therefore seems reasonable to believe that higher levels on the order of at least 10,000 lb per 1,000 cfs per day can be obtained. This would amount to 1.85 mg/l dissolved oxygen addition.

The cost of introducing oxygen by turbine aeration results from losses in turbine operating efficiency, and perhaps chiefly from the loss in effective throughput of water for power production purposes because of displacement of a part of the normal water flow by the volume of air introduced. The turbine, therefore, loses a proportionate amount of its power-producing capacity. The apparent gain in oxygen

TABLE III.—Summary of Power Data from Turbine Aeration

River and Location	Average Stream Flow (cfs)	O_2 Absorbed (lb/day/1,000 cfs)	Pounds O_2 Gain per kwh Loss		Per Cent Loss of Power Production	
			Observed	Calc.	Observed	Calc.
Flambeau (Pixley)	407	7,800	2.50	3.68	10.6	7.0
Wisconsin (Hat Rapids)	338	8,200	1.82	5.64	7.1	2.3
Wisconsin (Rothschild)	1,762	5,920	5.43	—	2.5*	—
Lower Fox (Kimberly)	1,219	1,890	3.30	—	6.0	—
Lower Fox (Rapide Croche)	975	7,100	4.07	—	8.7	—
Lower Fox (Rapide Croche)	1,043	6,153	3.20	3.93	10.1	—
Lower Fox (Little Rapids)	748	1,602	—	—	—	—

* Estimated.

per kwh lost was observed in the first 6 of the 7 studies carried out in 1958, and ranged from a low of 1.82 lb oxygen to a high of 5.43 lb oxygen per kwh lost. These observations of the apparent losses were obtained from readings of panel meters on the generators. However, hydraulic engineering authorities who have studied these data are agreed that the apparent loss observed is without doubt substantially larger than the actual losses of power production potential. This is especially true during low water periods when water which cannot pass through the turbine due to the admittance of substantial volumes of air added to the total throughput represents a saving of power production potential at a given gate opening. In at least two of the turbine aeration studies reported in Table III, attempts were made by the engineers to estimate correction factors which might apply. For the Flambeau River it was estimated that the actual amount of oxygen gain per kwh lost was increased from 2.50 to 3.68 lb, or a corrective factor of about 33 per cent. Similar calculations made for the tests at Hat Rapids on the Wisconsin River indicated that the pounds of oxygen gained per kwh lost was increased from 1.82 to 5.64 lb of oxygen. These latter data showed the indicated loss in power production potential to be one-third of the apparent loss. These four sets of data were recalculated to show that the percentage loss of power production by the turbine ranged from a high of 10.6 per cent for the observed data at Pixley Dam to a loss of 2.3 per cent from the calculated net loss in the tests at Hat Rapids on the Wisconsin River.

These questions concerned with the amount of power lost by a turbine and the cost of introducing a given amount of oxygen remain as problems to be further developed by later studies. There seems little doubt that the costs and the loss of power production are quite reasonable for the amount of work accomplished, but in spite of extensive discussion of the theory and practice with representatives of principal manufacturers of turbine equipment, consulting engineers, and university research authorities, it has proven difficult to formulate a reliable and common basis which could be employed for calculating these costs and losses. Each of the power dams and turbines under study have been suffi-

ciently different in design and operating characteristics as to make necessary individual studies of each location. Eventually, it is hoped that the fundamentals and variables affecting these questions will be subject for careful study under controlled conditions, probably with pilot models.

Discussion

Evaluation of the data derived from commercial-scale operation of turbine aeration at six locations on Wisconsin rivers during 1958 shows that this method of introducing oxygen to the waters of a flowing stream represents a practical way in which critical levels of low dissolved oxygen may be substantially alleviated. At the same time the data coupled with the closely controlled stream studies of prior years on these rivers indicate that the purification capacity of the rivers may be substantially increased during periods when the streams are organically overloaded.

The use of turbine aeration appears especially valuable for periods of hot summer weather when stream flows are low and the waste loading introduced to the stream from natural and man-made sources are higher than normal. Turbine aeration is not only low in its apparent operating costs, but where it can be applied through existing power dam installations, the capital charge for modifying existing equipment or for introducing proper design factors into new equipment is also quite low and may even be negligible. It is true that the opportunities to take advantage of turbine aeration are usually limited by the fortunate location of those power dam installations which happen to be available in the critical areas or other desired aeration stretches of the stream. Turbine aeration has to be utilized where the stream is low in dissolved oxygen content, on the order of 40-per cent oxygen saturation or less. Such critical areas are found some distances down-stream from the principal sources of pollution. There is no practical outcome to be expected from attempting to introduce oxygen to waters with high levels of oxygen saturation or upstream from the sources of stream pollution.

An additional and interesting benefit from turbine aeration has been under test on one Wisconsin river during the winter of 1958–59, where the level of dissolved oxygen had dropped to the critical point under low temperatures or extensive ice cover. The tests have progressed sufficiently far to make it quite apparent that turbine aeration may be of value for relieving and reducing the adverse effects of winter ice cover and low rates of oxidation prevailing in cold waters. It is well known that decrease in rates of oxidation may extend the stretches of critical low levels of oxygen far downstream from that observed during warm weather stream conditions. Power dams which may not be applicable for effective turbine aeration in summer months may in some cases be found to be well located for taking care of critical conditions developing during cold weather.

It may be well to add with regard to the cost of turbine aeration that data are accumulating to indicate that power losses and costs may be proportional to the operating hydraulic head for the turbine used. It is interesting to note that potentials for aerating under conditions of low hydraulic head appear practical, and that it may even be possible to use compressed air through turbines which are set at the level of the tail water and without suction head sufficient to draw atmospheric air into the turbine draft chest. However, it seems probable that the data yet to be developed by further tests in Wisconsin and elsewhere will eventually show that the costs will be relatively high at low hydraulic heads. Reported preliminary tests for turbine aeration at a high dam

with a head of several hundred feet have indicated that both the apparent and the calculated power loss were so small as to be negligible.

Still another benefit which seems likely to accrue from turbine aeration lies in the possibility for aerating the deoxygenated waters drawn from the stratified low levels of water in deep lakes and reservoirs. Some power dams located at the foot of deep reservoirs are already designed to draw water from the lower levels, and turbine aeration could be expected to overcome the lack of oxygen in the dead waters released from such impoundments. Redesign of intakes might eventually be justified where facilities are not already available for drawing off the stratified waters having low levels of dissolved oxygen.

Among other ideas proposed as alternative methods for introducing oxygen to river waters, consideration is often given to the possibility that oxygen gas might be employed with greater absorption efficiency than is the case with oxygen derived from atmospheric air. Under favorable conditions, tonnage oxygen can be made available at $20.00 per ton or less.

The data derived from the commercial-scale turbine aeration tests indicate the entire cost of dissolving oxygen from atmospheric air might be expected to range from about $3.00 to $15.00 per ton of oxygen actually absorbed when the base charge for power losses is rated at 1¢ per kwh. It, therefore, seems unlikely that purified oxygen gas can be used economically to replace air for turbine aeration installations, even at 100-per cent absorption efficiency. These calculations do not eliminate other potentials for using oxygen gas for direct aeration of rivers. The use of oxygen combined with a good method for dispersal may still be found to be relatively low cost and quite practical when compared with other methods of raising the dissolved oxygen content of river waters at locations where turbine aeration facilities are not available.

Future Developments of Turbine Aeration

Turbine aeration is expected to continue in operation during the summer of 1959 at all dams on Wisconsin rivers which are here reported, and plans have been made for new operations to be undertaken at several other power dams on the Wisconsin, Lower Fox, and Oconto Rivers. Outside of the State of Wisconsin tests are understood to be under way in Minnesota and Michigan. Commercial-scale tests are well advanced in recent studies by Wagner in Germany (7). Inquiries for information have been received from many other possible sources of river aeration in the United States and abroad. It is hoped that the various data can be brought together in the near future to help in further development of the design and operating problems observed and anticipated in turbine aeration.

Conclusions

1. Turbine aeration appears to be a practical means of adding oxygen to waters low in dissolved oxygen which are passing through suitably located and designed hydroelectric power installations.

2. Turbine aeration of rivers has been tested on full-scale power dam installations at six sites on three Wisconsin rivers in 1958.

3. Where turbines are so located as to be able to process oxygen-deficient waters (40 per cent saturation or less), practical amounts of oxygen were found to be absorbed at rates ranging from 1,600 to 8,200 lb O_2 per 1,000 cfs of stream flow per day at summer water temperatures in excess of 20°C.

4. Oxygen absorption efficiencies ranged from 15 to 35 per cent in five of these commercial-scale tests receiving extended study. A brief trial in-

dicated even higher absorption potentials at a sixth location.

5. Turbine aeration results in loss of power production, as judged by direct panel meter readings, from 2.5 to 10.6 per cent.

6. Power losses are indicated to be less than the observed readings because of saving of water stored for later use. The actual loss of power production is indicated to be one-third to two-thirds of the observed meter losses.

7. Average values of from 1.8 to 5.6 lb oxygen were found to be absorbed for each kwh of power loss.

8. If firm power is valued at 1¢ per kwh, the cost of aeration by this method appeared to be in the range of $3.00 to $15.00 per ton of oxygen absorbed.

9. Turbine wheels set well above tail water level will produce vacuum to draw atmospheric air into vents to the draft tube. Indications are that air may be successfully forced to turbine wheels which do not produce sufficient vacuum for full aeration potential.

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DIRECTORY OF ROTIFER STUDENTS

All students of the Rotifera in any fashion or any who use them as indicators or experimental animals are encouraged to send for a copy of "Second Preliminary List of Rotifer Students of the World." This list contains the latest list of rotifer workers. A copy will be sent free to workers in the field by writing to Dr. John J. Gallagher, 1716 East Terry, Pocatello, Idaho.

THE BIO-ASSAY APPROACH TO ESTUARINE POLLUTION PROBLEMS *

By Cedric E. Lindsay

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Much has been said about the extremely complex nature of biological problems connected with pollution work. This is especially true of marine waters where, instead of a one-way flow and a relatively constant volume of water, there are added the problems of tidal ebb and flood; horizontal and vertical mixing due to wind, temperature, salinity, and density. These forces may serve to dilute any waste very rapidly, or through stratification and low net removal serve to confine them to a relatively small volume of water. This discussion will be confined to one aspect of the estuarine environment—the shellfish.

One of the first problems facing a bio-assay worker is to determine the dilution of effluent present and where these wastes go. In an estuarine environment this requires repeated, extensive, and detailed hydrographic measurements which cover seasonal, and in some cases daily and weekly variations.

Recognition and Scope of the Problem

Once existence of a problem is suspected from routine field observations, the scope of purely biological monitoring must be expanded to determine whether undue mortalities, poor growth, poor fattening, or reproductive failures have occurred in shellfish. The percentage of mortality due to known predators or other factors must be recorded as well as numbers dead from unknown causes. If no assessable cause for the difficulty can be fixed and if the decline in numbers or condition appears abnormal for the particular area, the biologist must resort to bio-assays to pinpoint the effects of natural environmental factors and of industrial wastes present. The symptoms observed may be acute and thus lend themselves to short-term bio-assays or they may be of a chronic nature and be approached only through long-term, low concentration bio-assays.

Three types of bio-assays are necessary to permit bridging the gap between purely laboratory work and the opinions of the observer in the field. Procedures are known for use in applying laboratory bio-assays to specific effluents under carefully controlled conditions. Short-term effects of high and low concentrations of a particular industrial waste are perhaps the easiest to conduct and can be completed in a matter of hours or days.

Laboratory Bio-Assays

Another technique, originated by Lousonoff, bears mentioning in somewhat more detail. This one concerns the use of oyster or clam larvae as a bio-assay tool. These bivalve organisms are known to be highly sensitive to foreign materials in the water and in addition they have the advantage of a larval life usually less than 30 days and an egg development period of 48 hr. They are, in fact, so sensitive that development of suc-

* Presented at the 25th Annual Meeting, Pacific Northwest Sewage and Industrial Wastes Assn.; Spokane, Wash.; Sept. 25–27, 1958.

cessful culturing procedure has required a great deal of time and effort, since it is consistently necessary to produce spawning of the adult, successful egg development, normal growth, and good survival of larvae. The bio-assay is then conducted using control and test groups of larvae from a single spawning and subjecting them to various concentrations of the effluent under test. Criteria of effect on shelled larvae include survival, rate of growth, feeding activity, and per cent misshapen larvae. A 48-hr bioassay may also be made using per cent survival of the embryo from fertilization to formation of the larval shell. The percentage normal development in comparison to controls has been found to be an excellent means of determining relative toxicity.

Long-term effect of various concentrations of effluent can be done on various age classes of juvenile and adult animals in the laboratory. In the case of oysters, these may require from three to four years for a single bioassay since it is necessary to carry out research of this type through the entire life cycle. Concentrations used must be rigidly controlled so that the required levels can be maintained steadily at all times. As an example the group at the Washington Department of Fisheries Shellfish Laboratory near Quilcene is at present conducting research on the effects of sulfite pulp liquor on Olympia and Pacific oysters. Results of field hydrographic surveys have shown that there must be test concentrations ranging from 2 to over 100 mg/l, based on blow pit liquor containing 10-per cent total solids. The effects of these concentrations will not be measured merely in terms of survival or mortality, but in addition the effect on growth, feeding rates, histological conditions of the body organs, and development of reproductive material will be recorded and evaluated. At the termination of each part of the experiment, condition or fatness of test and control groups of oysters will be determined. The limitations of such tests as are outlined here center around the ability to supply adequate environmental conditions for the experimental animals.

Lagoon Bio-Assays

One method for relating results of laboratory tests to field conditions is by use of a lagoon which provides a semi-controlled natural environment. There is such a place next to the Quilcene laboratory. This 2½-acre lagoon is divided by a polyethylene membrane into two sections. In the initial test one side was polluted to 40 mg/l sulfite waste liquor and the control side was left unpolluted. During the experiment free water exchange with Hood Canal was prevented by keeping the tidal gates closed.

Initial groups of 400 Olympia oysters were introduced into each side; one week later 350 were placed in each side. One month after the beginning of the experiment, 100 oysters were removed from each side and at subsequent weekly intervals lots of 50 oysters were removed. In addition to regular weekly mortality checks, measurements of growth, mortality, fatness, and reproductive condition were made on each lot of oysters removed. The experiment was terminated after all detectable pollution had disappeared.

Field Bio-Assays

Field bio-assays are directly applicable to some types of problems such as those associated with acute toxicity. However, to have meaning they must be related to laboratory and lagoon studies. Probably the best type of field bio-assay involves the continuous measurement of environmental conditions over a long period of time. In the simplest terms it is a continuing ecological survey taken quantitatively at specific stations. Data are taken which will permit measurement and interpretation of the changes in the

environment. As an example, the Quilcene group is taking monthly measurements of fatness of oysters on a statewide basis; and of growth, fatness, and mortality in limited areas. Broad annual coverage is achieved by measuring growth, fatness, and mortality of all year classes present on every major oyster bed in the state. Monthly, annual, and intermittent data provide a picture of the changes in the oyster populations for the areas in each of the bays where observations are continued. It should be pointed out that of all commercially important organisms in the marine environment, clams and oysters are best suited for the field bio-assay. Their sedentary habits automatically fulfill three major bio-assay requirements; namely, ease of recovery for evaluation, assurance that the same individuals are being evaluated at all times, and inability to avoid, even for a short time (through migration), any of the factors affecting the environment.

Another example is afforded by work being done in one of the bays of southern Puget Sound. In that bay has been established a number of stations covering various tidal levels in all parts of the bay. Quantitative measurement of the changes in animal and plant populations is made at each of these stations, with particular reference to oysters and clams. At the same time, continuous data on water properties are maintained in this bay. Results of all field bio-assays will be compared to observed changes in climate, hydrographic conditions, bottom conditions, predation, disease, plankton, and industrial waste discharge.

Still another method by which field bio-assays may be carried out is by the use of floating and fixed live boxes which can be used for oysters, clams, and swimming fish; and by growth plots on the bottom which can be used for oysters and clams. The technique for conducting bio-assays of this type is actually very simple since it is merely necessary to maintain groups of animals of several year classes at various locations throughout the area to be tested. As a short-term bio-assay this technique can be used to measure acute toxicity. However, its chief value lies in the feature of maintaining stocks of identifiable test animals through protracted time periods. Of course, certain technical problems present themselves in maintaining and servicing the installations but these can be overcome. Analyses again involve growth, survival, and fatness of the test animals. In the area where these are being used there is also a continuing hydrographic survey. A major advantage of this type of work is that test animals are subject to a natural environment throughout its variations and may be expected to react similarly to those commercial species which are present in the area.

In conclusion, it is emphasized that to be of value, results of the bio-assays must be sufficiently sound to withstand scientific scrutiny. None of these techniques is considered adequate, by itself, to provide the necessary answers. Responsible biologists operate on the principle that detailed analyses and careful interpretation of laboratory, semi-controlled, and field bio-assays yield valid information on which intelligent decisions relating to waste disposal can be made.

SEE INSIDE FRONT COVER FOR ANNOUNCEMENT
OF THE NEW SEWER DESIGN AND
CONSTRUCTION MANUAL

THE OPERATOR'S CORNER

AUTOMATIC PROPORTIONATE SAMPLER*

By R. K. Snyder

Research Chemist, Division of Sewage Disposal, Cincinnati, Ohio

The Division of Sewage Disposal receives both domestic and industrial wastes for treatment in its plants. Due to the great diversity in volume and strength of the industrial wastes discharged to the city's sewerage system, these wastes require special consideration and examination. At periodic intervals the wastes from the various companies are tested to provide the necessary information on which to base surcharges and to determine if any dangerous or toxic materials are being discharged. To obtain samples for analysis, a crew of gaugers is sent to the manhole where the company's wastes enter the city sewer. There they set up a station and measure the flow over several consecutive 24-hr periods. The samples that they take at predetermined intervals have to be measured and composited according to the rate of flow. Each day's composited sample is sent to the laboratory for analysis.

It is obvious to anybody who has watched this procedure that if a suitable piece of equipment were available to automatically collect samples proportionate to the flow and composite them, a great deal of time and expense would be saved. To be sure, such machines are available, but they are designed to operate in open channels, not manholes. The shape of the average manhole may be compared to a milk bottle; the top opening usually is about 20 in. in diameter and the bottom diameter is commonly 48 in. Usually neither 110-v a-c nor compressed air is available. The dimensions of the manhole and lack of utilities preclude the use of any commercially made machines.

Description of Sampler

The decision was made that the Division of Sewage Disposal would attempt to design and build an automatic sampler. Ultimately, the sampler would be battery operated, but since a-c equipment is more common and easily purchased, the prototype instrument was to operate on 110-v a-c. The unit was to operate with a 90-deg V-notch weir. The levels of flow over this weir were to be limited to between 2 and 8 in. This meant the machine would sample over a flow range of from about 20,000 to 600,000 gpd.

Since the dimensions of a manhole seriously limit the length and width, the sampler is tall and narrow and can be easily lowered through a 20-in. circular opening. The framework which supports it in the channel and which permits careful adjustment in relation to the weir height, can be

* Presented at the 33rd Annual Meeting, Ohio Sewage and Industrial Wastes Treatment Conf.; Cincinnati, Ohio; June 17–19, 1959.

dismantled easily. The reason for this careful adjustment is that the sampling container which is lowered into the flow to pick up the sample must come to rest against its stops. All dimensions in relation to sample volume and flow have been calculated from this "stop" position. The sampling container is the essential part of the apparatus and is of a triangular shape as shown in the accompanying sketch (Figure 1). The general shape of the edges of this container as well as the actual dimensions are dependent on the size of sample desired, the depth of flow, and the type of weir with which it is to be used. The specifications for this particular container were mentioned earlier. In operation, the container supported by Pivot Rod *A* is lowered until Pivot Rod *B* rests in its notch (Stop 1) at the bottom of the sampler. The suspending action is gradually released, allowing the container to pivot on *B*, and the container, being properly counterweighted, leans over until Pivot Rod *C* hits Stop 2. The operating mechanism then reverses itself and as the container returns to a vertical position on Pivot Rod *B*, the sampling edge cuts the surface of the flow. It is at this point that the container picks up its sample and the size of it depends on the depth of flow. The upward motion is continued until Pivot Rod *C* encounters a cam track which causes the container to tilt and empty. A movable funnel is provided to guide the sample into a reservoir. This sampling container operates in a framework which supports the necessary guides, tracks, and sprockets. It is operated by a slow-speed gear-head motor, a suitable timer for lowering and raising the container, and two limit switches. The accompanying photograph shows the assembled sampler in the shop, the cover for the electrical assembly not being in place. This particular arrangement was for display and demonstration purposes (Figure 2).

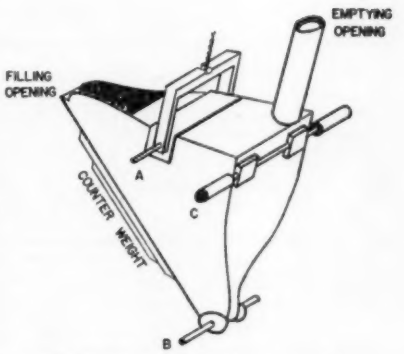

FIGURE 1.—Sampling container.

Test Results

When the sampling container was being designed, it was tested in a rectangular tank. The level of the water was adjusted to correspond to different levels of flow over a 90-deg V-notch weir. The volumes of sample picked up at the 3- and 4-in. levels were off somewhat more on a percentage basis than the samples from other levels, but the error was believed to be tolerable. With these preliminary tests finished, the sampler was thought to be ready for field testing.

The first field trials were run on July 29, 1958. The manufacturing plant selected for test purpose has a channel which is on ground level and enclosed in a small structure. The channel has a built-in 90-deg V-notch weir and there is a company-owned flow meter; also available was an electric outlet delivering 110-v a-c. Beyond these conveniences, it was simple to set up the sampler and have it continuously observed by several men. On each of the two test days the sampler was operated for 2 hr. Each sample collected was measured separately to evaluate sampler accuracy at various depths of flow. The results are shown in Table I.

The volume of samples the sampler delivered on the first day was 7.6 per cent higher than the theoretical volume; on the second day the delivered

TABLE I.—Test Results

Time	Actual Sample Vol. (ml)	Theoretical Sample Vol. (ml)	Depth of Flow* (in.)	Flow (mgd)			
				From Weir Table	From Meter	Computed From Actual Sample Vol.	Computed From Theoretical Sample Vol.
(a) First-Day Results							
12:30*	95	74	3⅜	.053	.08	—	—
12:45	220	200	5	.188	.20	—	—
1:00	223	200	5	.188	.21	—	—
1:15	270	250	5½	.237	.24	—	—
1:30	340	330	6¼	.309	.32	—	—
1:45	265	250	5½	.237	.26	—	—
2:00	275	250	5½	.237	.26	—	—
2:15	270	250	5½	.237	.25	—	—
2:30	300	280	5¾	.264	.28	—	—
Mean	—	—	—	.237	.25	.260	.241
(b) Second-Day Results							
9:20*	395	380	6½	.359	.36	—	—
9:35	415	400	6⅝	.376	.38	—	—
9:50	410	400	6⅝	.376	.39	—	—
10:05	395	380	6½	.359	.38	—	—
10:20	390	380	6½	.359	.38	—	—
10:35	355	340	6¼	.326	.33	—	—
10:50	415	400	6⅝	.376	.38	—	—
11:05	395	380	6½	.359	.38	—	—
11:20	375	360	6⅜	.341	.36	—	—
Mean	—	—	—	.359	.37	.378	.365

* Samples at 12:30 and 9:20 not included in means.

volume was 3.6 per cent higher. Incidentally, using this particular sampling container and rate of sampling, the mean 24-hr flow (in mgd) can be estimated by dividing the total 24-hr volume (in liters) by 100.

The automatic sampler was given a second field trial in the same test channel. The apparatus operated 24-hr/day on Sept. 22, 23, 24, and 25, 1958. Three samples were taken at 15-min intervals; one sample was taken by the automatic sampler and the other two were taken manually by the gauger. Of these two latter samples, one was taken in the usual way as the flow went over the weir, and the second was taken by submerging a container alongside the sampler for the same duration the scoop of the sampler was immersed in the flow.

The manually collected samples were examined to find out if there was any difference between the composition of the waste collected by the sampler and the waste collected by the gauger. The samples were collected over a 24-hr period, composited, and then taken to the laboratory where the total volume was measured. Each day's combined samples were homogenized and analyzed for suspended solids, BOD, and ether-soluble materials. The results of these tests and the calculated flows are shown in Table II.

No results are shown for flows or

volumes on Sept. 22 or 23 as it was impossible to bring the combined samples to the laboratory in the usual containers without spillage. On Sept. 24 and 25 the combined samples were transported in 5-gal carboys and there was no spillage except possibly in one instance as indicated in the table.

Again, the automatic sampler functioned nearly perfectly for the four-day period. There was difficulty with the counter at one time and this was corrected.

As in the first test, the volumes collected and the estimated flows are in good agreement. Since the time periods were much longer, the close relationship of the figures means more. The results of the test for suspended solids, ether-soluble material, and BOD are in fair agreement. The higher values obtained with the samples taken by the automatic sampler are difficult to explain.

TABLE II.—Comparison of 1958 Test Result

Item and Method of Sampling	Date			
	Sept. 22	Sept. 23	Sept. 24	Sept. 25
SS (mg/l)				
auto. sampler	680	650	385	360
manual at sampler	540	460	340	268
manual at weir	560	485	535	310
BOD (mg/l)				
auto. sampler	945	785	1210	1090
manual at sampler	780	760	1180	975
manual at weir	820	755	1130	975
Ether-soluble matter (mg/l)				
auto. sampler	136	178	138	130
manual at sampler	122	172	132	110
manual at weir	114	168	138	128
Volume (l)				
auto. sampler*	—	—	20.78	23.09
manual at sampler	—	—	19.95	22.86
manual at weir	—	—	19.42*	22.80
theoretical	—	—	20.39	23.28
Flow (mgd)				
auto. sampler	—	—	0.208	0.231
weir	—	—	0.203	0.230
meter	—	0.226	0.226	0.255

* Possible spillage.

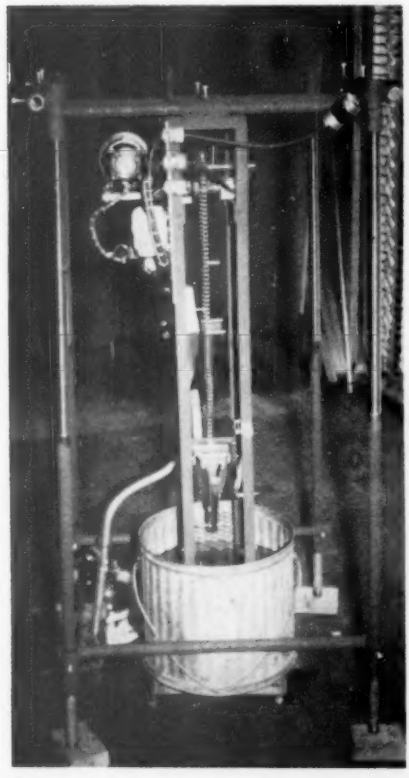

FIGURE 2.—Automatic waste sampler.

Summary and Conclusions

An automatic proportionate sampler for installation in manholes has been designed and built. Testing in the field has shown that it possesses sufficient accuracy for ordinary needs. It also seems feasible to design and build other sampling containers for different flows and weirs which would operate in the same framework. Battery operated samplers also should be built and tested.

Acknowledgment

Jack Arrington, Bob Ludwig, and Don Gehring in the Maintenance Department built the sampler. John Stanton and Malcolm Elam of Industrial Wastes supervised the testing of

the apparatus and Ralph Childers of the Laboratory was responsible for the analyses. The author wishes to thank these members of the Division of Sewage Disposal for their aid and suggestions for making this project possible.

SEWAGE WORKS MANAGEMENT *

By Ben Benas,† Walter B. Jones, and Keeno Fraschina

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This paper discusses management of the Bureau of Sewers and Sewage Treatment in San Francisco which is responsible for the collection, treatment, and disposal of sewage and the control of industrial wastes.

The problems of management of municipal sewerage works are many and complex. In addition to technical control, problems connected with personnel, fund control, equipment and material, work assignments, safety control, and public relations have to be solved. Supervisors, apart from having technical competence, must know the system, have administrative ability, be able to handle people, know accounting procedures, be able to meet the public and city officials, handle correspondence, and write reports.

In San Francisco there are special problems which may not apply to other communities or to industry. There are many old and poorly constructed pipe and brick sewers which must be maintained by the Bureau at higher-than-normal costs. The combination of hilly topography, combined sewers, and old sewers often creates emergency situations which must be dealt with instantly to protect public health and property.

Treating sewage at three plants increases treatment costs because there is duplication of personnel and there are more units to operate and maintain. Operation and maintenance must be adjusted to availability of funds which are allotted to specific functions. Some essential operations must be curtailed below desirable limits or deferred because of inflexibility in fund assignment or difficulty in obtaining additional funds. By the city charter, the work done by per diem and by some monthly employees must conform to union regulations. Work limits set by civil service regulations further limit flexibility in assigning or reassigning personnel. The system must be operated in a manner to satisfy both a public and press not always familiar with how the system operates, or what it does, or the restrictions imposed by fund limitation.

The Bureau of Sewers and Sewage Treatment is one of nine bureaus in the Department of Public Works. The Bureau is divided into three divisions. Treatment plants and control of industrial wastes are in the Sewage Treatment Division and pumping stations are in the Sewage Pumping Division. Activities pertaining to sewers are under the Sewer Division.

There are 248 regular employees in the Bureau, of which 106 are in sewage treatment, 13 in sewage pumping, and 129 in sewer maintenance and repair. To indicate the scope of Bureau activities, expenditures for 1957–58 totaled $1,930,000 of which $998,000

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† Formerly Superintendent, Bureau of Sewers and Sewage Treatment.

was for treatment plants and industrial wastes control, $107,000 for pumping stations, and $825,000 for sewer maintenance and repair. In addition, the Sewer Division spent $116,000 for side sewers paid for by property owners and $8,000 on work orders from other city departments.

Sewage Treatment Division

The three treatment plants are primary plants with a total dry weather flow capacity of 110 mgd. Treatment facilities include screening and grit removal, sedimentation, pre- and post-chlorination, separate sludge digestion, elutriation, vacuum filtration, and sludge drying. Treated effluent is discharged to San Francisco Bay and the Pacific Ocean.

The Division is headed by a senior engineer; each plant is supervised by a superintendent, a professional engineering classification. Each plant has two subsections, a laboratory and an operation and maintenance section; the laboratory is headed by a sewage-treatment chemist, a professional classification.

A chief operating engineer, an operating classification, is in charge of the operation and maintenance section. Within this section there are operating engineers who serve as shift leaders, one or more operating engineers for maintenance, and the required number of junior operating engineers. Laborers, truck drivers, and janitors report to the chief operating engineer for assignment.

The sewage treatment plants are, of course, always manned. The supervisory staff of superintendent, chemist, and chief operating engineer is on regular duty during the daytime on weekdays. Supervision on other shifts is delegated to the shift leaders under rules set by the supervisory staff.

The shift crews operate treatment units, collect samples, and make needed minor adjustments and repairs. Major repairs and maintenance are performed by maintenance crews consisting of an operating engineer, one or more junior operating engineers, a machinist, an electrician, and a painter. Services of machinists, electricians, and painters, as well as those from other crafts, are obtained from other bureaus and departments on work order. Engineering design for improvements is performed by the Bureau of Engineering, and landscaping is maintained by the Park Department using transferred funds.

The laboratory sections analyze plant samples, monitor receiving waters, and make investigations to improve plant operation. In addition, the sections analyze industrial wastes, test for pollutants in air samples, determine sources in leak complaints, and do work for other departments and agencies having need for the specialized knowledge and equipment available.

The superintendent sets over-all operating and maintenance policies utilizing laboratory and operating reports as guides.

Pumping Division

The Pumping Division operates and maintains 14 booster stations. The Division is headed by an assistant engineer, a professional classification, and under him are three operating engineers and nine junior operating engineers. The operating engineers supervise, operate, and maintain pumping stations in assigned districts. The junior operating engineers are on roving assignments except for three pumping stations.

Major repairs and maintenance requiring trained mechanics are done by other bureaus or departments on work order.

Sewer Division

The Sewer Division cleans sewers and catch basins, maintains and repairs main sewers and structures, installs and repairs side sewers, tests for ex-

plosive and other gases, and does other work related to the operation and maintenance of the sewerage system.

There are over 850 miles of main sewers, most of which are combined. Separate sewers are located mainly in low-lying downtown areas. Main sewers range in size from 6-in. vitrified clay pipe to three-compartment, reinforced concrete sewers with 7½- by 10-ft compartments. During rain, excess storm flow is diverted through 16 outfalls.

The Bureau superintendent acts as administrative head of the Sewer Division. Division operations are divided into construction and operation sections, each under an assistant superintendent of sewer maintenance and repair.

Construction Section

The construction section repairs and constructs main sewers and structures, does bricklayer work, and constructs and repairs side sewers. The work is in three work units designated main sewer, bricklayer work, and side sewers. Main sewer work and side sewer work are each supervised by a general foreman. Bricklayer work is directly supervised by the assistant superintendent.

Most material for field work is drawn from stores and all materials must be accounted for and charged to the proper job. Inventories of supplies are watched and replacements ordered when minimum stock levels are reached.

Main Sewers

Cribbing crews excavate, lag, and backfill main sewer trenches, and in the construction or repair of structures, they cut pavement and do miscellaneous work. Excavation is by hand in sandy soil and where the facilities of other utilities demand extra caution. A mechanical digger has recently been acquired for other excavations.

Pipe repairs are mainly required on the clay sewers. Breaks occur from scour, open joints, settlement, or poor house connections: scour is prevalent on streets with steep slopes serving previously undeveloped areas; open joints and settlement occur mainly in old sewers; and poor house connections are found on connections made by private contractors before regulations required city installation or inspection.

Two cribbers usually constitute a crew which is supplemented occasionally with additional cribbers or laborers.

Bricklayer Work

Bricklayer crews repair main brick sewers, construct and repair manholes and catch basins, lay pipe over 8 in. in diameter, and do bricklayer work required for the Bureau of Street Repair.

Underground crews have one bricklayer and two hodcarriers. Crews on structures and pipe laying have one bricklayer and one hodcarrier.

The 76 miles of brick sewers constructed before 1906 require continual maintenance. Most deterioration results from failure of the lime mortar originally used and from scour.

Side Sewers

Side-sewer crews excavate, lag, lay pipe, and backfill side sewer trenches. Two cribbers normally constitute a crew. The number of crews employed depends on the number of side sewers being built or repaired. At present, only two crews are motorized. The trucks carry sufficient tools, equipment, and material to complete a job and move without delay.

The side sewer from the building to the main sewer belongs to the property owner. By city ordinance, all installations and repair work under city streets on side sewer connections must

be performed by the Bureau except when side sewer construction is included with main sewer construction under public contract. Before any work is started, the property owner makes a deposit to cover the estimated cost of construction or repair of the side sewer from the curb to the sewer. Final payment covers only actual costs.

Operation Section

The operation section cleans sewers and catch basins, responds to complaints, flushes sewers, assists in construction, and tests sewers for explosive gases. The work is in four work units designated: sewer cleaning, eductor operations, servicing complaints, and gas detection. One general foreman supervises service crews and eductor operations; the assistant superintendent supervises sewer cleaning and gas detection.

Sewer Cleaning

Cleaning crews clean main sewers, sumps and tanks at pumping stations and treatment plants, do miscellaneous work in connection with sewers, and service compaints during rains when service crews cannot handle all calls.

Sewers are normally cleaned by buckets dragged between manholes. Other methods employed are manual cleaning of large sewers, flushing, and manual and mechanical rodding of pipe sewers.

Eductor Operations

Eductor crews clean catch basins, pump out basements and sumps, pump sewage during main sewer repairs, wet down trenches when hydrants are not accessible, and do other related work.

Catch basins in San Francisco are not simply street inlets but separate structures holding about 10 cu ft of liquid. The solids collected are disposed of at city dumps.

Normally, one-reel eductors are used. A two-reel eductor, first of its type in the west, was placed in operation in Aug. 1956 to clean catch basins on both sides of one-way streets and alleys while traveling in one direction; it is used primarily in downtown areas. Another unit of this type was placed in service in 1958.

Servicing Complaints

Service crews respond to complaints, wet down main and side-sewer trench backfills, flush sluggish or temporarily plugged sewers, and do emergency work.

Each crew has a compartmentized, covered, one-ton truck fully equipped with hoses, lights, barricades, rods, and special tools. Four crews are on regular day work; one is on night work.

As examples of work done in a typical year, about 3,800 calls are made on side sewers, 1,600 on main sewers, 1,100 on catch basins and manholes, and 2,000 on servicing warning lights and barricades, investigating seepage, and other work.

Gas Detection

The gas detection crew tests sewer manholes, pumping stations, treatment plants, and other structures for explosive gas, carbon monoxide, hydrogen sulfide, and oxygen deficiency; checks for specific industrial wastes in sewers; assists in safety training; and checks condition of manholes and sewers. Natural gas leaks are reported to the gas utility company, and testing is continued until the leak is corrected. Clogged sewers, loose bricks, and sludge deposits in manholes are reported to other sections.

The gas detection unit is on night operation in May to test manholes in downtown areas inaccessible during the day.

Almost without exception, the presence of methane in sewers is the result of leaks in natural gas mains or services.

Miscellaneous

Large sewers, particularly those in areas where trouble is anticipated, are regularly inspected to forestall serious difficulties. A crew is assigned on a part-time basis to inspect other sewers and structures prior to any extensive field work. Recently, the inspection of sewers has been expanded to include photographing of pipe interiors and other locations difficult to enter. This program of photographic inspection shows promise of being of great value.

Yard service at the main maintenance yard to receive deliveries, keep equipment in working order, check materials and supplies on hand or on order, and maintain an inventory of city tools and equipment assigned to field crews has been found to be essential.

Management Problems

Fund Procurement and Control

With minor exceptions, Bureau funds are budget appropriations from general city funds. Budget estimates are made by each division for salaries, overtime, holiday pay, temporary salaries, wages, contractual services, materials and supplies, and equipment. The budget request for each item states the need and details to substantiate the request, and lists expenditures for the past five years and for six months of the current year. The requests are reviewed by the Director of the Department of Public Works, the Chief Administrative Officer, Mayor, and Board of Supervisors. The Department of Public Works budget can be reduced but not increased by subsequent reviewing agencies. These reviewing agencies give primary consideration to past expenditures, sometimes without sufficient regard to current needs and inflation. Thus, the need for clear and detailed records to substantiate budget requests becomes apparent. In the case of treatment plants which are relatively new, budgetary recognition must be given to the fact that increased maintenance is required as the equipment gets older. This is a factor sometimes not recognized by non-technical budget-processing agencies, and any records which will substantiate maintenance needs will be of great value.

Budgeted funds are generally allotted quarterly by the city controller. The controller does not normally approve quarterly allotment increases without proof of necessity and a statement that the yearly budget appropriation will not be exceeded. Additional funds must be obtained by supplemental appropriation by the Board of Supervisors. It is difficult for an operating bureau to adjust expenditures to quarterly periods because requirements are affected by seasonal factors.

Funds are set up for each division by specific categories, and are not interchangeable. Expenditures in the Pumping and Sewer Divisions are controlled by the Bureau superintendent. In the Sewage Treatment Division, each plant is allocated specific amounts and the superintendent is held responsible for keeping expenditures within these limits. In particular, attention must be paid to expenditures for overtime, temporary employment, some contractual items, and materials and supplies.

The official accounting records are kept by a separate bureau in the Department of Public Works. As delays in reporting make control of expenditures difficult, duplicate records have to be kept by each division. This is now being corrected by new and revised procedures in the accounting bureau using automatic machines. Nevertheless, it will still be necessary for this bureau to continue to keep approximate current records of expenditures for planning and control.

Personnel

The number of job classifications and work limits for each classification

are set by civil service rules which, for working classifications, conform substantially to union rules. The number of employees in each classification is limited by ordinance.

As a result, there is lack of flexibility in assignment or reassignment when employees are off for sickness, vacation, or other reasons. Existing rules require an employee to notify the Bureau in advance of expected absence. However, this is not always done or possible. Civil service procedures do not permit relief employees to be hired on short notice. As a result, in the Sewage Treatment and Pumping Divisions, overtime must be arranged or a shift crew must work short. In the Sewer Division, crews must be reassigned or other work found for members. To minimize malingering, a Department of Public Works doctor checks employees reported sick.

During vacations, the Sewage Treatment and Pumping Divisions must obtain replacements for shift operators. In the Sewer Division, personnel must be shifted to make sure a minimum number of full crews are working on each function. The Bureau has formulated definite vacation rules which give consideration to seniority and employee requests. Scheduling vacations well in advance has been found helpful.

In assigning personnel, it is necessary to consider personality and ability in forming shift and field crews. Some factors to be considered are: Is a man a leader or follower? Is he dependable? What is his physical condition? Is he safety minded? Is he a hard worker? Does he observe rules? Has he ability for independent thinking? Does he get along with others? Although there are exceptions, some general rules may be formulated: (*a*) Put followers with leaders, (*b*) new employees with experienced employees, (*c*) physically weak with strong, (*d*) undependable employees with strong leaders, and (*e*) in some cases, nonconformists with nonconformists.

The problem of training new employees is a difficult one which has not been solved completely. If possible, new employees are assigned to work with experienced personnel. This has not always been possible, and there have been occasions where equipment has been damaged by inexperienced operators.

The ideal way to train personnel for sewage pumping and treatment would be to have sufficient positions budgeted so new employees can be assigned to supervised maintenance work long enough for them to become thoroughly familiar with operations.

It would be very helpful if the promotional ladder under civil service were arranged to keep trained employees within the department rather than allowing promotion to other departments.

One point that should not be overlooked in training employees who come in contact with the public is the matter of public relations. Employees should be cautioned against making irresponsible statements which might mislead, misinform, or cause embarassment to the department.

The maintenance of discipline is a duty of management. If discipline is not continuously and uniformly maintained, the morale of the entire organization will suffer and the quality and quantity of work will deteriorate. The means of enforcing discipline will, of course, differ from place to place. In San Francisco, discipline is maintained by means of explicit rules established by civil service.

Sewage Treatment and Pumping Division Work Assignment

In the Sewage Treatment and Pumping Divisions, first consideration must be given to the number and location of stations, functions performed, and physical layout. Operators must be stationed at key locations where

emergencies can be observed and acted on promptly. There is, of course, a limit to the number of functions and the area which can be covered by an operator. In large plants with several operating stations as at San Francisco, it is necessary to have a supervisor on each shift who is responsible for overall operations and who can assist or relieve station operators. Easy interplant communication is helpful and in many cases, necessary. In addition to regular duties, each shift operator should be given definite maintenance assignments for which he is responsible.

In the treatment plants, laborers are employed on day shifts for general cleaning and to perform other unskilled duties which allow shift operators to devote maximum time to operation and maintenance.

The chief operating engineer plans maintenance and requisitions replacement parts. Maintenance should be scheduled for minimum interference with operation.

In the laboratory sections, the chemist determines sampling and testing procedures and programs research projects. Routine analysis is performed by one or more laboratory technicians. The chemist also acts in place of the superintendent in his absence. In setting sampling programs, the chemist must consider whether tests are essential to plant operation, whether they will improve plant operation or reduce cost, and whether regulatory agency requirements are being met.

To control plant operation, the supervisory staff must review plant and laboratory logs daily, physically observe operation and plant condition, and inspect work done. Plant operating costs should be reviewed at frequent intervals.

Sewer Division Work Assignment

Except for emergencies, all sewer maintenance and repair work is programmed before field assignments. The first step is to determine the kind and amount of work necessary to maintain the system, and then to determine the composition and number of crews to do the work. For administrative purposes and for efficient use of men and equipment, it has been found best to separate construction from operation.

Main sewer repairs are scheduled first at sewer breaks or where there is a possibility of property damage or danger to public health. Local courts rule that prior knowledge of sewer defects makes the city responsible for damages. Remaining work is scheduled on a city-wide priority basis. A record is kept showing the date and source of information, location of break, size of sewer, and probable extent of work required.

Underground bricklayer work is first scheduled at breaks and in sewers known to be in bad condition, otherwise by districts. Other bricklayer work is assigned daily.

In side-sewer work, repair is given first priority. Side-sewer construction is usually scheduled by date of fund deposit, but the side-sewer foreman may consider building occupancy and construction needs in assigning priority.

In sewer cleaning, emergencies take priority over all other work; other work generally is programmed by districts.

In eductor operations, catch basin cleaning is programmed by districts. Emergencies and special work are assigned without regard to district.

Complaints are serviced in order of receipt unless severity of the condition warrants other action. Service crews also have regular assignments in known trouble areas.

Gas detection in manholes is normally scheduled by districts, but testing of known leaks is continued without regard to district until the leak is corrected.

Service, eductor, and gas-detection crews and side-sewer cleaning and construction crews report daily to the maintenance yard. All other crews report directly on the job and their equipment is left as close to the job as possible to increase available working time in the field.

Providing field services at other than regular day shifts presents problems. It is too costly to have crews working at all hours; therefore, except for impending inclement weather, one service crew is on duty from 4 PM to midnight, Monday through Friday, and one employee on the Saturday and Sunday day shifts. Any calls received requiring service at other hours are given to a general foreman on call at home. He investigates, determines what needs to be done, and calls out field crews if necessary. He is paid for the time worked but not for standby services. Consideration is being given to include standby services with the job requirement and to set compensation accordingly. Getting men to work overtime is sometimes difficult, as overtime work is not compulsory. Men, if at home and available, are selected by seniority and reliability.

Field supervision of a large number of small crews presents a problem since supervisors must be in the office part of the day to assign work, order materials, and check daily reports. Another problem is that employees of some field crews all have the same rank which makes it difficult to assign crew responsibility.

Daily field records must be kept showing date, employee classification, location, work performed, time required, and weather conditions for work control and cost accounting purposes and for protection against legal action. Forms for field reporting must be simple and clear to obtain meaningful records. Use of a machine card system is being considered. Cards, however, will not be of value unless a very simple coding system is worked out because field personnel find it difficult and distasteful to keep accurate records. It will still be necessary to maintain daily job records not possible to include on machine cards for performance analysis by supervisors and for use in legal actions.

Records are kept of monthly purchases of materials and supplies by unit, and the records are checked against past usage and current needs to control expenditures, determine if the use of any item is unreasonable, schedule future purchases, and prevent depletion of essential stock items.

Field supervisors should spend as much time as possible on direct field supervision. Work performance analyses and map recording should be done by staff employees. These records should be analyzed by function and crew to indicate over-all efficiency and to point out where corrective measures should be taken or methods changed.

Equipment

Equipment should always be available and in good operating condition. To accomplish this, sufficient spare parts and adequate repair facilities must be provided.

Spare parts or equipment items for mobile units used regularly by sewer cleaning and eductor crews should be on hand. Spares of items not readily available or needed for emergency work should also be carried in stock. In addition to central shops for major repair and overhaul, a mechanic should be available to minimize downtime and prevent major breakdown.

Old, obsolete, or worn equipment should be replaced when downtime is excessive or repair costs too high. Adequate records are essential to substantiate budget requests for replacements.

First cost, depreciation, upkeep, ability to perform, over-all cost, and operator pay and classification are factors affecting procurement of new equipment. In San Francisco, certain classes of equipment require special

operators. There must be sufficient productive work to keep these operators working continuously. When equipment is used infrequently, it may be more advantageous to rent it than to purchase it.

Equipment records are kept on each item. These records describe the item, give essential data, show the work done on maintenance and repair, and the time the unit is out of service. The records are checked to determine if costs are excessive, if the equipment is not performing satisfactorily, or if the unit is not being properly operated.

Management should be alert to possible application of new developments in equipment and automatic processes. Major factors to be considered are whether substantial lowering in overall costs or number of employees is possible. In applying automatic processes to sewage treatment, it is well to remember that operators often credit these processes with undue reliability and thus to neglect necessary checking.

Safety

The prevention of accidents is a matter recognized as one of the more important functions of management. Accidents are not only disagreeable from the humanitarian point of view but are downright hard on the budget. If the supervisor is not safety-minded, it is not likely that safety-mindedness will spring forth full blown in the minds of the people he supervises. Recognizing this fact, all supervisory personnel in the Department of Public Works have been required to attend a course of study in accident prevention. This course was arranged by the school department in cooperation with the National Safety Council.

The value of such a course of instruction remains long after the details have been forgotten. The major value lies in the safety-mindedness which is instilled in the minds of the participants, who, in turn, affect others by example.

Regular safety meetings are held in the treatment plants and the Sewer Division as well as at bureau and department levels. At these meetings, each accident is analyzed for the purpose of preventing repetition. It has been found that most accidents are caused by unsafe acts rather than lack of safety measures by management.

The purchase of clothing for employees was recently reviewed. By state law, any item considered essential for safety can be obtained with city funds.

Summary and Conclusions

From the foregoing, it can be surmised easily that conditions in municipal sewerage works are continually changing. Thus, those charged with management responsibility must be forward looking and alert. In this article problems associated with management in San Francisco have been stated.

It may be concluded that for successful operation of municipal sewage works, management must concern itself with operating results; with people and what they do, as greatest expenditures are for personal services; with control of other expenditures; with maintaining plant and equipment; and with changes and improvement.

Because of the wide scope of activities and skills needed, this is a fertile and challenging field.

Acknowledgments

The Director of the Department of Public Works of the city of San Francisco is Mr. Reuben H. Owens. The operations of the Bureau of Sewers and Sewage Treatment are under the more immediate observation and control of Mr. Lawrence J. Archer, Assistant Director.

LIMITATIONS OF SEWAGE TREATMENT PLANTS IN HANDLING INDUSTRIAL WASTES *

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At any meeting of a sewage and industrial wastes association, the subject of discharging industrial wastes into municipal sewers can usually provoke a controversy. There is no disputing the problems and difficulties that may arise from such discharges. The disagreements, however, can be greatly reduced if a serious attempt is made to understand all sides of the problem and to obtain a common viewpoint with the "opposition."

Viewpoints

Sewage plant operators and board members of municipal sewerage authorities, when they have time to think about the matter, recognize that industrial plants are important to the community, that they are heavy taxpayers, and that they have certain rights to the use of the sewerage system. It is also conceded that industries inevitably produce wastes, and must discharge them somewhere. In urban areas, the municipal sewer may be the only place for such discharge. Even if other drainage channels are available, these may transport the wastes directly into local streams, a misfortune that the sewerage system is specifically designed to prevent. Industrial wastes must, therefore, be expected in sewers, and the sewage treatment plant must make provision to handle them.

Industry people, on the other hand, must recognize that the sewage treatment plant operator has very definite responsibilities to his community. And there is no denying that it is industry's duty to cooperate in the field of sewage treatment, just as in law enforcement, fire prevention, and other civic activities.

Industry's cooperation with the sewerage authority involves utilizing the sewer system in a reasonable manner, without damaging the facilities by such use. Industries must not discharge into the municipal sewers wastes that will damage city property, including both sewers and equipment at the treatment plant. They must not discharge wastes that are hazardous to the community, or that interfere with treatment plant processes and operations. If the wastes of industry cause undue consumption of chemicals at the treatment plant, or an unusual amount of labor, the industry involved should be willing to pay a special fee to cover this added cost. And finally, industry must not expect the municipal treatment plant to purify wastes which it was never designed to purify.

If industrial chemists, engineers, and management can achieve a common viewpoint with sewerage commissioners and sewage plant operators, each will recognize the requirements of the other. Nearly always, successful operation of the industrial plant and the treatment plant will result.

In the following pages some of the specific difficulties that must be reconciled are reviewed, and an attempt is made to determine the responsibility for each.

Hazardous Discharges

The discharge into municipal sewers of materials that are hazardous to people—residents of the community and treatment plant personnel—

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should never be tolerated. It is a requirement on industry that such possible discharges be eliminated completely from the sewer system and disposed of by other means.

Flammable liquid wastes, such as gasoline and other organic solvents, are a hazard in sewer systems, and must be prevented from entry. Such liquids usually are not miscible with the sewage, but float on the surface, forming stagnant pools and giving rise to flammable and explosive vapors. The elimination of such wastes, by suitable traps, is a responsibility of industry. The sources of such wastes include not only large petroleum and chemical plants and metal-finishing industries, but also small cleaning operations, garages, and service stations.

Another flammable and explosive waste that requires careful disposal is spent carbide slurry from acetylene generation. Such a waste might continue to produce acetylene after its discharge into the sewer, with obvious hazard. Most municipal sewer ordinances, reasonably enough, totally prohibit the discharge of acetylene waste.

Toxic gases and vapors, in large sewers and wet wells, can be dangerous to people who must enter these chambers for inspection, cleaning, or repair. Hydrogen cyanide, carbon monoxide, hydrogen sulfide, and wastes that may release these gases, are among the discharges that require careful control.

Damage to Physical Plant

It is also apparent that wastes which are damaging to the sewer properties should be kept out of sewers. The sewerage system, including the collecting sewers and the treatment plant, is community property, and must not be subjected to the possibility of damage by any individual or any industry.

Corrosion is a well-recognized type of damage, often attributed to excessive acid discharges by industry. Acid wastes will attack concrete or iron structures, including pipe, pumps, manhole fittings, and even treatment plant equipment if the acidity should penetrate so far. Fortunately domestic sewage is normally slightly alkaline, and adequate dilution of acid wastes with sewage will neutralize the acidity. Corrosion may also result from non-industrial causes, as from hydrogen sulfide formed in septic sewages or sludge deposits. But industry should neutralize or equalize its acid wastes to a harmless pH before discharge.

Solid wastes should be kept out of municipal sewers as far as practicable. Heavy sludges and sediments may clog sewers and render them inoperable. At the least, more frequent cleaning is required, and this is expensive. Greases and heavy oils are particularly troublesome for this reason, and because they clog equipment at the treatment plant. There is no excuse for the discharge of coarse solids such as rags and sticks; these are damaging to pumps and other equipment, and can easily be kept out of sewers by simple measures. In general, all industries should intercept readily removable solids before the discharge of waste.

Damage to Plant Processes

One of the most aggravating problems of the average sewage plant operator is the upsetting of his treatment processes by some unanticipated and often unrecognizable variation in the raw sewage flow. No part of his plant is immune from such occurrences, although most troubles seem to occur in the aerobic biological processes, the final clarifiers, or the digesters. Sometimes industrial wastes are the cause of such disturbances, and often industry is blamed whether guilty or not. The wise operator is less anxious to proclaim blame than to investigate his troubles scientifically and to identify their cause, which may often be within his own plant or within the range of

variations he must normally expect in incoming sewage.

There are, however, some types of industrial waste which are known to interfere with treatment plant processes. These should be eliminated at the industrial source in order that the treatment plant may perform its intended purpose. Chemical substances like acids, cyanides, or metal ions are upsetting to biological processes if they occur in too high a concentration. Just what the permissible concentration is for any particular substance or combination of substances is not easily determined, but a considerable literature has been built up already and research work is continuing. Each industry should become familiar with the toxic effects of the chemical materials it handles; and each treatment plant operator should acquaint himself with the toxic substances that may exist in the industries of his area.

Plant processes are subject to interference by excessive discharges of oils, especially heavy oils and greases. These not only clog sewers, as mentioned previously, but also affect the operation of pumps, comminutors, and similar equipment, and disturb biological action by coating the microorganisms. Oily substances are deleterious to aerobic processing, and may also affect sludge digestion.

Wastes of excessively high temperature may cause damage to sewers by softening bituminous sealing materials, and may cause septicity in sewers by accelerating decomposition reactions. Too much heat in wastes received at the plant may interfere with settling and with the control of biological activity.

Overloads

Industry may at times place a greater demand on the sewerage system than it is capable of handling, thus interfering with efficient operation. Hydraulic overloads (too much water) cut down detention times in sedimentation tanks and other units, thereby decreasing plant efficiency. If such overloads occur intermittently in the manufacturing plant a controlled-flow discharge into the sewer may solve the problem. If the overload is clean water, it could be diverted to a storm sewer or to natural drainage. If the overload is continuous and polluted, an increase in sewer and treatment plant capacity may be necessary unless the industry can provide its own treatment and disposal.

An overload of settleable solids may cause clogging of the sewer lines, especially of wet wells at pumping stations. If the overload reaches the plant, it places an added burden on settling tanks, digesters, and sludge disposal facilities. Perhaps this is no handicap to an adequately designed treatment plant; but if it becomes a burden or if the industry wishes to be helpful, such wastes can be settled before discharge to the sewer.

High concentrations of organic matter can often be handled by the sewage treatment plant; if so, this is the most satisfactory means of disposal. If the treatment cost is increased unduly, as for chlorine or other chemicals, the industry involved should be prepared to pay a service charge. In some plants additional equipment has been required, such as a second trickling filter, and the industry has paid for its capital cost as well as for operation. Of course, industry always has the privilege of removing the overload from its waste ahead of the sewer. If this can be done by improved housekeeping and simple salvage, it is worth while; but if actual treatment is needed the sewerage authorities may be able to provide it more economically. Of course organic overloads that are not amenable to treatment, such as oils, or those which interfere with treatment, such as cyanides, must be separated by industry and kept out of the sewers.

Shock Loads

One of the frequent causes of poor operation of the sewage treatment plant is the occurrence of shock loadings in the incoming sewage. Industry must recognize that sewage treatment processes are geared to continuous 24-hour, 7-day operation, and that the more nearly uniform the raw material entering the plant, the smoother the operation of the plant and the better the quality of its product. Most treatment plant operations have a considerable capacity to absorb fluctuations in raw material—this is necessary because even domestic sewage flow varies in volume and concentration from hour to hour. But this leveling-out ability of the plant must not be abused by shock loads beyond its capacity. This is one reason that many communities provide separate storm sewers—the sudden rush of relatively pure water from a quick rainstorm is itself a shock to the treatment plant. Consider then the unbalance that may result if the shock discharge consists of acids or chemicals, sediment, or high temperature, rather than just a high rate of flow.

The prevention of shock discharges is primarily the responsibility of industry, and can usually be accomplished by simple means. Of course some expense is involved, but it is generally minor in comparison with elaborate treatment works. Perhaps the sudden discharge can be avoided back in the manufacturing process itself by considering this as a factor in plant scheduling. If manufacturing operations will not permit so simple a solution, it may then be desirable to install holding or blending tanks at some point between the manufacturing and the discharge into the sewer. Such equalization, if properly planned, may completely eliminate the problem of shock loads.

To some degree, of course, industry is entitled to look to the sewerage system to furnish equalization. The sewer pipes, manholes, and wet wells provide places where holding and blending of the wastes and sewage can take place. The prudent engineer, however, will rely on this factor only for a final leveling, after providing most of the equalization within the boundaries of the industrial facility.

It is proper to consider what degree of shock-load removal is necessary; this is different for each sewerage system and for every industrial plant on the system. Obviously a small sewage treatment plant will be more seriously disturbed by a given shock discharge than a large treatment plant. Obviously an industry located near the treatment plant will find its shock discharges transmitted to the plant with less equalization in the sewers than an industry near the end of the sewer line; although such a plant has the compensation that more sewage is available for blending and dilution.

Permissible shock discharges in different communities will vary because of the type of sewage treatment provided. A treatment plant that gives only the primary treatment of sedimentation is affected by peak flows (shortened detention time in the plant) and by heavy sediment concentrations (overload of settling equipment), but is not particularly disturbed by soluble chemicals or toxic materials or even radioactive wastes. A secondary treatment plant or a plant with sludge digestion, both involving biological treatment, may be seriously hampered by shock loads of any nature.

Untreatable Materials

Some industry people, and other citizens too, take the attitude that once a waste is discharged into the sewer, their responsibility is at an end. This implies of course that the waste is acceptable in the sewer, that it is not hazardous nor destructive of property, and will not affect operation of the treatment plant. Most sewage plant

operators would probably agree, under these limitations.

However, one must also consider whether or not the treatment plant provides adequate treatment before discharge to the receiving stream. The plant is intended to remove sewage-type solids and sewage-type organics, and to destroy pathogenic microorganisms, but it may have little or no purifying effect on such components as brines, cyanides, fluorides, phenols, color, detergents, or radioactivity. A secondary treatment plant does a better job than sedimentation alone, but even after secondary treatment the quantity of these components discharged to the river may be excessive and pollutional. If an industry has wastes like these (and other examples will come to mind), it should consider whether the dilution with municipal sewage plus such removal as the treatment plant provides is adequate to eliminate stream pollution. If not, the problem is basically industry's, although the sewerage authority may be able to help.

Conclusion

The municipal sewer is the logical place, sometimes the only place, for the discharge of industrial wastes. To the extent that these wastes can be processed in the sewerage system, without hazard or damage or interference with the treatment of other wastes, they should be welcomed. If the wastes are not acceptable to the sewer, it is industry's duty to make them so, by special forms of pretreatment ahead of the sewer. With good cooperation between industry and the sewer authority, and with mutual understanding of the problems involved, industrial waste disposal can be accomplished with a minimum of friction. This should be the goal of all.

WHAT AN OPERATOR NEEDS TO LEARN *

By T. R. Haseltine

Secretary, Certification Board, Pennsylvania Sewage and Industrial Wastes Association †

The Pennsylvania Sewage and Industrial Wastes Association has had a voluntary plan for the certification of sewage treatment plant operators since 1954. The plan is administered by a five-man board consisting of one man appointed by the Dean of Engineering of The Pennsylvania State University, one man appointed by the Chief Engineer of the Sanitary Engineering Bureau of the Pennsylvania Department of Health, and three men appointed by the Pennsylvania Sewage and Industrial Wastes Association. It is mandatory that at least two of these men be active operators of sewage treatment plants.

Rules and Regulations

The rules and regulations under which the plan is administered are in part as follows:

> For the purpose of this certification plan an operator is defined as any person actually employed at one or more sewage treatment plants who, in the opinion of the Certification Board, has a substantial degree of responsibility for the operation and maintenance of that plant or plants and so must be capable of making decisions as to policy and directing the work of subordinates. As such, the term "operator" may include plant superintendents,

* Presented at 31st Annual Conference, Pennsylvania Sewage and Industrial Wastes Assn.; University Park, Pa.; Aug. 12–14, 1959.

† Mr. Haseltine is a Partner in the firm of The Chester Engineers, Pittsburgh, Pa.

assistant superintendents, chemists, chief operators, responsible shift operators, or others who are regularly required to translate the results of laboratory tests into plant operating procedure or otherwise exercise considerable independent judgment in the control of the plant operation. However, subordinate employees who have no other duty than to carry out specific orders issued to them by their superiors, or members of the mechanical or construction trades whose only responsibility is the repair or maintenance of mechanical equipment are excluded. Also excluded are such officials as Mayors, Municipal Managers or Engineers, Directors of Public Works, Factory Managers, or Factory Superintendents who may exert some broad general supervision of budgetary or other policy matters, but who do not actually direct the daily routine operation of the plant. Consultants, Professional Engineers, or representatives of equipment manufacturers who occasionally advise and recommend methods of operating any or all portions of one or more treatment plants, but who do not actually direct their daily routine operation, are also excluded.

Plant Size

The rules provide for the issuance of both regular and temporary certificates. All certificates issued specify the maximum capacity and type of sewage treatment plant for which its holder is qualified to operate. As regards capacity, sewage treatment plants are classified on the basis of their design capacity into five categories as follows: (a) less than 0.1 mgd, (b) less than 1.0 mgd, (c) less than 7.5 mgd, (d) less than 20.0 mgd, and (e) more than 20.0 mgd.

Plant Type

As regards type of treatment, all sewage plants are classified under one of the following categories: (a) primary, (b) chemical precipitation, (c) biological filtration, and (d) activated sludge.

Holders of regular certificates for types of treatment other than primary treatment are permitted to operate primary treatment plants of the same size-classification without other certification. Holders of regular certificates for activated sludge treatment are permitted to operate biological filtration plants of the same size-classification without other certification.

Minimum Requirements

Concerning minimum requirements for certification, the rules are in part as follows:

> Certificates will be issued by the Board on the basis of its opinion of the applicant's knowledge and responsible experience in actual operation of sewage treatment plants. In order to qualify for either a temporary or a regular certificate, the applicant must be able to read, write and perform arithmetic calculations necessary to keep satisfactory operating records, which will be deemed to be the equivalent of a grammar school education and constitutes the minimum educational requirement for such certification. Further education should aid an operator in understanding and operating sewage treatment processes and hence reduce the actual operating experience necessary to qualify for regular certification.

The information in Table I is contained in the rules and regulations for the guidance of the Board in evaluating an applicant's eligibility for regular certification. The rules further state that in making such an evaluation the Board shall take into consideration any correspondence or short courses the applicant may have taken as well as his experience in occupations closely paralleling one or more phases of sewage treatment plant operation. If the Board is undecided concerning the applicant's knowledge or responsible experience in sewage treatment plant operation, it shall examine the applicant.

In the absence of sufficient experience for regular certification, the Board may, at its discretion, issue a temporary certificate. Holders of temporary certificates may apply for regular certificates after one year of actual operating experience, and if they fail to make application for regular certification within a period of two years after assuming such operating duties,

TABLE I.—Experience Requirements for Certification

Maximum Capacity of Plant (mgd)	Type of Treatment	Minimum Years of Experience as an Actual Operator Normally Required for Applicants Having Varying Degrees of Education		
		Grammar School Graduate	High School Graduate	College Degree in Appropriate Scientific Field
Less than 0.1	Primary	2	1	0
	Chemical precipitation	5	2	0
	Biological filtration	3	1	0
	Activated sludge	5	2	0
Less than 1.0	Primary	4	2	0
	Chemical precipitation	6	3	1
	Biological filtration	5	2	0
	Activated sludge	8	4	1
Less than 7.5	Primary	6	3	1
	Chemical precipitation	8	5	2
	Biological filtration	7	4	2
	Activated sludge	9	6	3
Less than 20	Primary	8	5	2
	Chemical precipitation	10	7	4
	Biological filtration	9	6	3
	Activated sludge	12	8	5
More than 20	Primary	10	7	4
	Chemical precipitation	12	9	6
	Biological filtration	12	8	5
	Activated sludge	15	10	7

or if the Board sees fit to refuse such applications, the temporary certificates become null and void.

Examinations

During the first four years of its administration, the Board required neither written nor oral examinations and during that time issued 98 regular certificates and 34 temporary certificates. Since March 1958, the rules and regulations have been amended to make a written examination mandatory for any change in certification. Examinations are held twice a year. The first examination was held in August 1958 concurrent with the Association's Annual Meeting at The Pennsylvania State University; only a few applicants took the examination. The second examination was held in February 1959 in the various regional offices of the State Health Department. A total of 49 men took the examination. This paper attempts to summarize the results of the February tests.

The examination was divided into two parts: the first dealt with primary treatment and was compulsory for all applicants; the second was optional and dealt with secondary treatment. One set of questions was issued to those desiring certification for biological filtration and another set was issued to those desiring certification for activated sludge plants. A total of 24 men took the activated sludge examination and 14 took the one for biological filtration. The examinations were written and applicants were permitted to use any handbooks, text books, and personal notes they desired.

Admittedly, the examination was difficult and small-plant operators had to answer the same questions as big-plant operators. This plan was pur-

posely followed to give an opportunity to the operators of small plants to demonstrate their qualifications for certification to larger plants. Unlike most scholastic examinations, however, no predetermined passing grade was established. It developed that many of the applicants did not understand this procedure or realize that the Board intended to simply use the examination results in conjunction with its review of the applicant's experience in reaching an over-all evaluation of his qualifications for certification. Therefore, considerable consternation developed among some of the operators over the difficulty of certain problems they were asked to solve.

The examination for primary treatment consisted of 33 multiple-choice questions, 14 of which were designed to test the applicant's general knowledge of primary treatment processes and equipment. The other 18 could not be answered without making some arithmetical computations.

The examinations for secondary treatment also were multiple-choice questions. Those for activated sludge contained 33 questions of which 20 required some arithmetical calculations. Those for biological filtration contained 33 questions of which 16 required arithmetical computations.

Applicants were given about two hours in which to complete each part of the examination and they were permitted to skip as many questions as they wished. Papers were graded on the basis of (a) the percentage of correct answers to all questions, and (b) the percentage of correct answers to the number of questions attempted. They were similarly graded on the general questions and the arithmetical questions separately. Almost invariably the grades were far higher on the general questions than on the arithmetical ones. Many of the applicants attempted only a very few of the latter and not a single applicant answered all of them correctly. Therefore, it appears that in general operators are very weak on arithmetic.

Results

The examination results showed that when given the essential laboratory data, only 39 per cent of the applicants could correctly compute the concentration of suspended solids in a sample of sewage, only 61 per cent could correctly compute the concentration of volatile suspended solids in the sample, only 34 per cent could correctly compute the solids content of a sample of sludge, and only 31 per cent of the applicants for secondary treatment certificates could correctly compute the BOD of a sewage sample. The number of applicants failing to attempt solutions to these four questions were 10, 17, 15, and 36 per cent, respectively.

All applicants for secondary treatment certificates were questioned about the changes that take place in the alkalinity of sewage sludge as it undergoes digestion. Only 50 per cent of them guessed correctly, and "guessed" is used because only 30 per cent had any idea what the alkalinity would be in a primary digester of a properly functioning, two-stage digestion process, and only 22 per cent could correctly calculate the alkalinity of a sludge sample when given the essential laboratory data. Although only 8 per cent of the applicants skipped the first of these questions, 31 per cent skipped the second, and 36 per cent skipped the last.

All of the remaining arithmetical problems dealt with actual operation rather than laboratory data. When given the flow through a primary basin and the suspended solids concentration of the influent and effluent, 44 per cent of the applicants could correctly compute the weight of solids removed and only 3 per cent skipped the problem. However, when given similar data and also the volume of wet sludge pumped from the basin, only 41 per cent of

the applicants could correctly compute the solids concentration of the sludge, and 32 per cent did not attempt the problem at all.

In another problem, the volume and solids content of a sludge before concentration and its volume after concentration and decanting were given. Only half of the applicants could correctly compute the solids content of the concentrated sludge and 15 per cent did not attempt the problem.

When average analyses of fresh and digested sludge were given, only 17 per cent of the applicants could correctly compute the percentage reduction in volatile matter resulting from digestion although all but 19 per cent attempted the problem. When the same data plus the daily average volume of fresh sludge entering the digester was given, only 15 per cent could correctly approximate the average daily gas production and 29 per cent did not attempt a solution.

A group of four questions common to both secondary-treatment examinations dealt with vacuum filtration of sludge. The data given included the sq ft of filter surface, the time of operation, the volume of wet sludge applied, and its solids content as well as that of the resulting filter cake. The total weight of wet sludge applied to the filters was correctly calculated by 33 per cent of the applicants, but only 11 per cent could correctly calculate the weight of filter cake removed; however, 28 per cent correctly calculated the weight of sludge solids removed and 31 per cent correctly calculated the filtration rates in pounds per square foot per hour. Percentages of applicants skipping these four questions were 49 for the first three and 50 for the last.

About 61 per cent of the biological-filtration applicants had a working knowledge of how the capacity of a centrifugal pump varies with its speed, but when given the static head and the total discharge head at one rate of pump delivery, only 38 per cent could correctly calculate the total discharge head at a higher rate of pump delivery. Both questions were skipped by 23 per cent of the applicants.

When the flow through a plant was given in mgd and the output of the blowers was given in cfm, only 44 per cent of the activated sludge applicants could correctly express air consumption in cu ft/gal, and 18 per cent skipped the question entirely.

Sixty-one per cent of the activated-sludge applicants could correctly calculate the Mohlman or sludge volume index when given the ppm of suspended solids in the mixed liquor and the volume of sludge in a liter graduate of that liquor after 30 minutes' standing. but when the sludge volume index and the solids concentration were given, only 39 per cent could calculate what volume the sludge would occupy in a liter graduate after standing 30 minutes; 13 and 22 per cent, respectively, failed to attempt these questions.

When given the flow of sewage, the flow of return sludge, and the concentration of solids in the mixed liquor along with certain other data such as the BOD of the influent to the aeration tank, the BOD of the final effluent, and the sludge volume index, only 9 per cent of the activated-sludge applicants could correctly calculate the solids concentration of the return sludge and 82 per cent did not attempt to answer the question.

When given the rate of wasting sludge to a thickener in gpm, and its solids concentration in ppm, 61 per cent of the activated-sludge applicants could correctly compute the weight of solids wasted in 24 hr, but when they were also given the concentration of sludge pumped from the thickener, only 43 per cent could calculate the weight of solids removed and only 26 per cent could calculate the average rate of pumpage from the thickener.

TABLE II.—Experience and Educational Background of Certificate Holders

Type of Certificate Issued		Years of Actual Operating Experience	Educational Background of Recipient
Maximum Size of Plant (mgd)	Type of Treatment		
Over 20.0	Activated sludge	2	B.S. in chem. eng.
Over 20.0		9	B.S. in civil eng.
Less than 20.0		2½	M.S. in san. eng.
		14	8th grade
	Bio. filtration	12	10th grade
Less than 7.5	Activated sludge	4	High school grad.
		1½	High school grad.
		2½	8th grade
	Bio. filtration	7	High school grad.
	Primary	1½	High school grad. plus 1 yr civil eng.
	Primary	4	High school grad. plus 1 yr bus. col.
Less than 1.0	Activated sludge	1½	7th grade
		1	B.S. in forestry
		5	High school grad.
		4	High school grad.
		5	10th grade
		5	High school grad.
		4	8th grade
		3	High school grad.
		3½	High school grad. plus 1 yr arch. drafting
		2	10th grade
	Bio. filtration	2	A.B. in English plus 2 yr in science & math.
		2	11th grade
		8 mo	High school grad.
		1	High school grad.
		4	High school grad.
		4	High school grad. plus 1 yr bus. col.

Certificates Issued

After review of the examination results as well as the experience and educational qualifications of the applicants, the Board saw fit to issue 35 regular and 6 temporary certificates. Two men were refused certification because they could show no actual experience as an "operator" as that term is defined in the rules and regulations previously quoted. No action could be taken in the case of two other applicants because of their failure to file records of their experience and educational qualifications.

In the opinion of the Board, two who already held regular certificates did not do well enough in their examination to warrant any change in the type or size of plant for which they were certified and two men who held unexpired temporary certificates were refused regular certificates because of their poor showing in the examination. Two operators of primary treatment plants did so well in their examinations that they were issued certificates for secondary treatment plants. Two operators of secondary treatment plants did so well in their examinations that they were issued certificates for plants of larger capacity than those at which they were trained or employed. Fifteen of the regular certificates that were issued were for the same type of plant but of smaller capacity than those at which the applicant was trained or employed.

Possibly this is an indication that the examination was too difficult or too severely graded; however, when the types of regular certificates issued are compared with the educational background and experience guides as set forth in Table I, it appears that 17

men received certificates for either larger plants or higher types of treatment than they would have qualified for without examination and conversely only 8 men did so poorly in the examination that they received certificates for either lower types of treatment or smaller plants than their educational background and experience would have indicated that they were entitled to. The remaining 14 recipients of regular certificates were unaffected by the examination, i.e., they received the same type of certificate as their educational background and experience would have qualified them for had they not been required to take the examination.

In the Board's opinion, the results of the examination showed that although a higher education is helpful to a sewage treatment plant operator, it is by no mean essential to the operation of a moderate-sized plant. Table II shows the experience and educational qualifications of the recipients of the top 27 regular certificates issued. For each size and type of plant the recipients are listed in the order of their showing in the examinations.

TIPS AND QUIPS

Pennsylvania's Industries Treat Wastes *

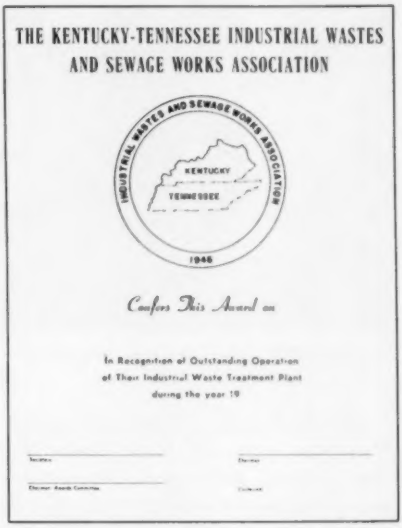
By the end of 1958, 1,235 of the 1,743 waste-producing industries in Pennsylvania had waste treatment facilities. This amounts to 71 per cent of the total.

Ventilated Storm Sewer †

When Dulles International Airport is completed, it will have ventilating fans in the storm sewer to keep explosive air-fuel mixtures from accumulating. Located about 20 miles from Washington, D. C., the airport will occupy more than 15 square miles.

Kentucky-Tennessee Awards

Shown to the right is a picture of an award of the Kentucky-Tennessee Industrial Wastes and Sewage Works Association which recognizes the outstanding operation of an industrial waste treatment plant. Similar awards recognize: (*a*) outstanding sewage treatment plant operation, (*b*) the industry or community which has done the most to abate a serious pollution problem, and (*c*) the member who has written the best news article for the Kentucky-Tennessee Newsletter.


THE KENTUCKY-TENNESSEE INDUSTRIAL WASTES AND SEWAGE WORKS ASSOCIATION

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1946

Confers This Award on

In Recognition of Outstanding Operation of Their Industrial Waste Treatment Plant during the year 19

Kentucky-Tennessee award

Four awards for operation, four for pollution abatement, and one for news reporting can be made each year.

* From *Clean Streams* (June 1959).

† From *Building Science News* (Nov. 1959).

Plastic Sanitary Sewer

From E. A. Symonds, Superintendent of Sewers, Needham, Mass., comes word on what is believed to be the first 8-in. plastic sewer main * in the United States.

FIGURE 1.—Fifteen-foot lengths of eight-inch plastic pipe.

The particular job consisted of 300 ft of 8-in. main and three house services. Each service connection consisted of approximately 50 ft of 4-in. plastic pipe. The installation was completed in June 1959.

One of the interesting features of plastic sewer main is that any desired length can be furnished since the extrusion of the pipe is a continuous process. The manufacturer suggested 30-ft sections, but the practical considerations of handling from yard to job site and the limitation on the total length of ditch to be opened at any given time made 15-ft lengths desirable (Figure 1). With a weight of 2½ lb/ft, each 15-ft length weighed only 38 lb. Two lengths were easily carried by one man.

Installation

Installation was handled by a Boston contractor who received only verbal instructions before proceeding with the work. By having an experienced sewer contractor make the installation it was hoped to get a better comparison of the problems involved in the use of plastic pipe.

The contractor started laying pipe from an existing brick manhole in which an 8-in. vitrified clay stub had been installed previously. The plastic pipe was inserted into the stub, the joint sealed with jute, and finally filled

FIGURE 2.—Joint for plastic pipe is solvent welded.

* Manufactured by the Evanite Plastic Co., a Division of the Evans Pipe Co., Ulrichsville, Ohio.

with cement mortar. To insure against leakage the whole joint was encased.

The pipe laying presented no special problem even though considerable ground water was present. A generous amount of crushed stone was used as a bed; the stone was banked on each side of the pipe at least to the spring line. Backfilling to a depth of about one foot above the top of the pipe was carefully done by hand and the rest of the trench was filled by machine.

The lengths of pipe were joined by plastic cement. The pipe, with coupling, was lowered to the trench bottom and cemented into the previously laid coupling (Figure 2). After a length of pipe was jointed in the trench, it was adjusted by the use of washed stone to conform to the proper line and grade and then covered with selected backfill.

House service connections were made with saddle-type plastic wyes with 4-in. branches. A drill and keyhole saw were used to cut the necessary hole in the plastic main. Plastic cement was then applied to the inside surface of the plastic 8- by 4-in. saddles and to the plastic main around the hole. The saddle was then held against the main (Figure 3) until the cement hardened (this method of making connections is extremely convenient after the main has been covered, but on initial installations in the future, sleeved plastic wyes fabricated from pipe stock will be used to eliminate the cutting of the oval hole in the main).

The actual cost of laying and joining the 15-ft lengths of 8-in. main did not differ materially from that experienced with 13-ft lengths of asbestos-cement pipe, but it was markedly lower than the cost of installing 4-ft lengths of clay pipe. According to Mr. Symonds, the greatest installation advantage offered by plastic pipe is that it can be cut anywhere and rejoined without further machining or working. It is cut easily with a handsaw, it can be drilled to make connections, and any inspection hole cut in the main can be closed merely by cementing a piece of pipe over the hole.

A minor objection to the pipe furnished was that it had too much longitudinal flexibility. This necessitated very careful bedding and backfilling to maintain grade and alignment. This condition was reported to the manufacturer who has since altered the plastic formulation to achieve greater rigidity.

FIGURE 3.—Four-inch plastic house connection joins eight-inch main.

After completion of the installation, the pipeline was found to be completely straight and free of any moisture. Several days later, the downstream manhole was checked and found to be dry.

Evaluation After Six Months of Service

After six months, the main was again checked for evidence of deformation and leakage.

A mirrored light was flashed from one manhole to the other. The pipe was found to have retained its original alignment and roundness. There was no infiltration at joints.

Since the initial installation of a plastic sanitary sewer, an 8-in. plastic storm sewer has been laid with equal success.

News and Notes

Of Persons and Activities in the Wastewater Field

Clarence E. Keefer has assumed the duties of Sewerage Engineer, Bureau of Sewers, Baltimore, Md.

O. C. Thompson, Union Carbide Chemicals Corporation, has been elected Chairman of the Water Pollution Abatement Committee of the Manufacturing Chemists' Association.

LeRoy W. Van Kleeck, former Principal Sanitary Engineer of the Connecticut State Department of Health, has joined **Bowe, Albertson & Associates,** New York.

R. F. Orth, General Manager of the **Johns-Manville Pipe Division,** has been elected a Senior Vice-President of the company.

Edward F. Morey, Jr., is now heading the newly organized Western Regional Office of **B-I-F Industries, Inc.,** with headquarters in Dallas.

A. V. (Cap) DeLaporte, Director of the Division of Laboratories and Research for the Ontario Water Resources Commission, recently retired after serving 48 years with the Ontario government.

John W. Cunningham, Consulting Engineer of Portland, Oregon, died recently. He served a partial term as Vice-President of the Federation during 1955–56.

George W. Kelsey, Vice-President of **B-I-F Industries, Inc.** of Providence, R. I., in December was installed as President of the Water and Sewage Works Manufacturers Assn.

Parsons, Brinckerhoff, Hall & Macdonald, engineering firm of New York, has changed its name to **Parsons, Brinckerhoff, Quade & Douglas.**

Eugene Weisberg has been named Director of Sanitary Engineering for Pantech Engineers with headquarters in Pittsburgh. He was formerly Assistant Research Professor of Sanitary Science of the Graduate School of Public Health at the University of Pittsburgh.

Robert S. Ingols, Head of the Department of Applied Biology at the Georgia Institute of Technology, has been elected Chairman of the American Chemical Society's Division of Water and Waste Chemistry for 1960.

Michael A. Groen is now Plant Supervisor of the Bromley sewage treatment plant, Kenton-Campbell Counties, Ky. He has been Superintendent of Sewage Treatment at Dearborn, Mich., for 25 years.

Thomas J. Powers, Supervisor of Waste Control for the Midland, Mich., Division of the Dow Chemical Company, has been named to head the consulting section of Dow Industrial Service, a new division.

Louis R. Howson, Senior Partner in the Chicago firm of **Alvord, Burdick & Howson,** has been named the "Chicago Civil Engineer of the Year, 1959" by the Illinois Section of ASCE.

L. D. Matter has joined the staff of the Philadelphia firm of **Albright & Friel, Inc.** and will have headquarters in Harrisburg, Pa. He was formerly Chief Engineer of the Water Supply Section of the Pennsylvania Department of Health.

H. J. N. Kessener, inventor of the brush aerators bearing his name, died recently in The Netherlands.

Smith and Loveless, Inc., Lenexa, Kans., has become a division of Union Tank Car Company, Chicago. B. Alden Smith and Compere Loveless will continue to direct the operation of the new division.

Wildred F. Langelier, Professor Emeritus of Sanitary Engineering of the University of California, received a 50-year membership certificate from the American Chemical Society. Professor Langelier has been recognized widely for his research in water chemistry.

Paul O. Richter, Vice-President of **The Eimco Corporation,** has been named General Manager of the Filter Division, in Salt Lake City, Utah.

Frank M. Macioce of Park Ridge, Ill., has been appointed Regional Sales Manager of Industrial Filter and Pump Manufacturing Co.

Charles C. Nolan has been named Western District Manager for **Hardinge Company, Inc.,** and will be in San Francisco. **R. L. Baldwin** is retiring as District Manager but will continue to act in a full-time consulting capacity.

William T. Ingram, New York consulting engineer, has opened a West Coast office in Walnut Creek, Calif.

Jones, Henry & Williams, Toledo engineering firm, has occupied new headquarters at 2000 West Central Ave.

BIOSORPTION® Activated Sludge Plant
Neillsville, Wisconsin

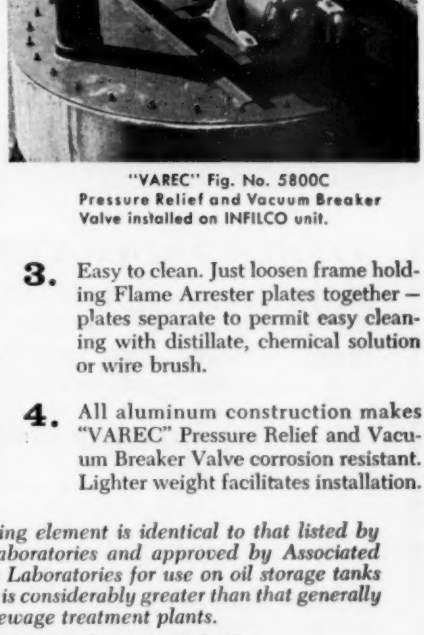
Consulting Engineers
Davy Engineering Co., Inc
La Crosse, Wisconsin

4 SUPERIOR FEATURES

"Varec"

PRESSURE RELIEF and VACUUM BREAKER VALVE with FLAME ARRESTER

FIG. No. 5800C



"VAREC" Fig. No. 5800C
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1	Safety in Wastewater Works	$0.75	$1.50
2	Utilization of Sewage Sludge as Fertilizer	0.75	1.25
3	Municipal Sewer Ordinances (Revised)	—†	—†
4	Chlorination of Sewage and Industrial Wastes	1.00	1.25
5	Air Diffusion in Sewage Works	1.00	1.25
6	Units of Expression for Wastes and Waste Treatment	0.25	0.50
7	Sewer Maintenance	1.00	1.50
	Multiple wire binder for above manuals	2.00	2.00
8	Sewage Treatment Plant Design	3.50	7.00
9	Design and Construction of Sanitary and Storm Sewers	3.50	7.00
	Uniform System of Accounts for Sewer Utilities (revised)	—†	—†

OTHER FEDERATION PUBLICATIONS

INDEX, SEWAGE WORKS JOURNAL, Volumes 1–20, 1928–1948 (Subject, Author, and Geographical), buckram, $3.00.

INDEX, SEWAGE AND INDUSTRIAL WASTES, Volumes 21–30, 1949–1958 (Subject, Author, and Geographical), buckram, $4.00; paper, $3.00.

GLOSSARY—WATER AND SEWAGE CONTROL ENGINEERING, paper cover, $1.00.

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Proceedings of Member Associations

NEW ENGLAND SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

Spring Meeting

The 1959 Spring Meeting of the New England Sewage and Industrial Wastes Association was held at Exeter and Durham, N. H., May 13–14, 1959. Total registration was 149.

The technical program included papers on the Pease Air Force Base sewage treatment plant, treatability of industrial wastes, financing, maintenance of chlorine equipment, and protective coatings.

A field trip to the Pease Air Base treatment plant followed the technical sessions.

Annual Meeting

The 31st Annual Meeting of the New England Sewage and Industrial Wastes Association was held at the Sheraton-Biltmore Hotel, Providence, R. I., Oct. 21–22, 1959. Total attendance was 203, including 13 ladies.

The technical program opened with a joint session on "Rebuilding of a Major Sewage Treatment Plant" by Ariel A. Thomas, Metcalf and Eddy, Boston, Mass., and Howard F. Carpenter, Chief of Sanitation, Providence, R. I., covering the Providence plant.

Following the joint session, concurrent sessions were held. An operator's session covered "Meter Maintenance" by David Giammarco, B.I.F. Industries, and "Maintenance of Conveyor

(*Continued on page 79a*)

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pumps almost everything
(what will go in will come out)

secret: recessed impeller; continuous open passage
(chances are it will work profitably for you)

Western Machinery Company 650 Fifth Street • San Francisco, Calif.

WATER POLLUTION CONTROL PRODUCT GUIDE

This list of products and services is offered as an information aid. The list was derived by solicitation from those firms listed. The JOURNAL will make every effort to keep this list current with the help of members and advertisers. All Associate Members and all advertisers for the past calendar year are included. Many of the services and products are advertised in this issue. See page 95a for an alphabetical listing.

Aerators (*also see* Diffusers, Air)
Alpha Ltd. (Switzerland)
American Well Works
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Graver Water Conditioning Co.
Infilco Inc.
Link-Belt Co.
Pacific Flush Tank Co.
Penberthy Mfg. Co., Div. of Buffalo-Eclipse Corp.
Permutit Co., Div. of Pfaudler-Permutit Inc.
Smith & Loveless, Inc.
Walker Process Equipment Inc.
Wemco Div., Western Machinery Co.
Yeomans Brothers Co.
Zimmer & Francescon

Agitators
Link-Belt Co.

Air Cleaning Equipment
Westinghouse Electric Corp.

Air Compressors (*also see* Blowers, Compressors, and Fans)
Chicago Pump Co.
Fairbanks, Morse & Co.
Komline-Sanderson Engr. Corp.
Worthington Corp.
Yeomans Brothers Co.

Air Diffusers (*see* Diffusers, Air)

Asbestos Cement Pipe Products
Filtration Equipment Corp.
Industrial Materials Co.
Johns-Manville Sales Corp.
Keasbey & Mattison Co.

Ash Handling Equipment
Link-Belt Co.

Automobiles and Trucks
Vulcan Materials Co.

Bearings
Link-Belt Co.

Bins, Storage (*see* Tanks)

Biologists (*see* Directory of Engineers)

Blowers (*also see* Air Compressors, Compressors, and Fans)
Chicago Pump Co.
Roots-Connersville Blower, Div. of Dresser Industries Inc.
Yeomans Brothers Co.
Zimmer & Francescon

Boilers
Combustion Engineering, Inc.

Building Maintenance Equipment and Supplies
Homestead Valve Mfg. Co.

Cast Iron Pipe Products
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
Industrial Materials Co.
U. S. Pipe & Foundry Co.
R. D. Wood Co.

Cement (*see* Concrete)

Centrifuging Equipment
Bird Machine Co.

Chains
Chain Belt Co.
Link-Belt Co.
Webster Mfg., Inc.

Chemical Feed Equipment
Builders-Providence Div., B-I-F Industries
Eimco Corp.
Fischer & Porter Co.
Graver Water Conditioning Co.
Infilco Inc.
Jeffrey Mfg. Co.
Komline-Sanderson Engr. Corp.
Link-Belt Co.
Omega Div., B-I-F Industries
Permutit Co., Div. of Pfaudler-Permutit Inc.
Proportioneers Div., B-I-F Industries
Wallace & Tiernan Inc.

Chemicals
Cloroben Chemical Corp.
Dow Chemical Co.
General Chemical Div., Allied Chemical Corp.
Tennessee Corp.
Vulcan Materials Co.

Chemists (*see* Directory of Engineers)

Chlorination Equipment
Builders-Providence Div., B-I-F Industries
Everson Mfg. Co.
Fischer & Porter Co.
Graver Water Conditioning Co.
Wallace & Tiernan Inc.
Zimmer & Francescon

Clarifier Equipment (*also see* Sedimentation Equipment)
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Chain Belt Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Graver Water Conditioning Co.
Hardinge Co., Inc.
Infilco Inc.
Jeffrey Mfg. Co.
Komline-Sanderson Engr. Corp.
Lakeside Engineering Corp.
Link-Belt Co.
Permutit Co., Div. of Pfaudler-Permutit Co.
Walker Process Equipment Inc.
Yeomans Brothers Co.
Zimmer & Francescon

Clay Pipe Products
Industrial Materials Co.
National Clay Pipe Mfrs., Inc.
Wedge-Lock Clay Pipe Mfrs., Inc.

Coatings and Linings (Pipe and Tank)
Amercoat Corp.
Centriline Corp.
Inertol Co., Inc.
Koppers Co., Inc.
National Water Main Cleaning Co.
Perry-Austen Mfg. Co.
Pittsburgh Coke & Chemical Co.
Stebbins Engr. & Mfg. Co.
Vulcan Materials Co.

Comminutors (*also see* Shredders and Grinders)
Alpha Ltd. (Switzerland)
American Well Works
Chicago Pump Co.
Dorr-Oliver Inc.
Gruendler Crusher & Pulverizer Co.
Infilco Inc.
Jeffrey Mfg. Co.
Smith & Loveless, Inc.
Walker Process Equipment Inc.
Worthington Corp.

Compressors
Chicago Pump Co.
Fairbanks, Morse & Co.
Komline-Sanderson Engr. Corp.
Roots-Connersville Blower, Div. of Dresser Industries Inc.
Worthington Corp.
Yeomans Brothers Co.

Computers
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.

Concentrators (*see* Sludge Concentrators)

Concrete
Portland Cement Assn.
Vulcan Materials Co.

Concrete Pipe Products
American Concrete Pressure Pipe Assn.
American-Marietta Co.
Gray Concrete Pipe Co., Inc.
Industrial Materials Co.
Lock Joint Pipe Co.
Portland Cement Assn.
Price Bros. Co.
Vulcan Materials Co.

Construction Equipment
Chain Belt Co.
Eimco Corp.
Worthington Corp.

Controls (*also see* Instruments, Recording and Control)
Bailey Meter Co.
Builders-Providence Div., B-I-F Industries

Sludge Collectors'' by C. E. Smith, Chain Belt Company.

The consultant's session heard a panel discussion of ''Accelerated Sludge Digestion Systems'' by Frank Burdee, Dravo Corporation; Robert L. Yocum, Dorr-Oliver Inc.; Thomas Forrest, Chicago Pump Company; and Leonard L. Langford, Pacific Flush Tank Company.

After lunch, the Mayor of Providence brought greetings from the City, and Federation Secretary Ralph E. Fuhrman commented and reported on Federation affairs.

Also, he presented the 1959 Hatfield Award for excellence in sewage plant management to Clarence V. Hickey, Blackstone Valley Sewer District, Providence, R. I.

The 1959 Bedell Award for service to the local association was presented to Walter M. Kunsch, Waterbury, Conn., for his excellent work on the Salary Study and the Operator Certification Committees.

The following officers were elected to serve during 1959–60:

President: Walter M. Kunsch, Waterbury, Conn.
1st Vice-President: Joseph B. Hanlon, Quincy, Mass.
2nd Vice-President: Howard F. Carpenter, Providence, R. I.
Secretary-Treasurer: Earl R. Howard, Hartford, Conn.

EARL R. HOWARD
Secretary-Treasurer

CANADIAN INSTITUTE ON SEWAGE AND SANITATION

The 26th Annual Convention of the Canadian Institute on Sewage and Sanitation was held at the Royal York Hotel, Toronto, Ont., Sept. 28–30, 1959. The total registration was 671.

(*Continued on page 81a*)

SLIDE GATES by FILTRATION *

These slide gates are fabricated of "Chemstone E". This is an asbestos-cement sheet with an acid resistance impregnation which is moulded at high pressure. Surfaces are sanded to a smooth finish.

SEND FOR FREE LITERATURE and Specifications

We consider this material more suitable than metals since it is relatively light in weight, corrosion resistant, acid resistant, does not require painting, and has a lower frictional coefficient than most metals after a period of use. This enables gates to be more readily removed.

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273 HOLLENBECK ST. ROCHESTER, N. Y.

Burgess-Manning Co., Penn Instruments Div.
Chicago Pump Co.
Fischer & Porter Co.
Foxboro Co.
General Electric Co.
Homestead Valve & Meter Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
Permutit Co., Div. of Pfaudler-Permutit Co. Inc.
Rockwell Mfg. Co.
Wallace & Tiernan Inc.
Westinghouse Electric Corp.
Worthington Corp.

Conveyors
American Well Works
Chain Belt Co.
Chicago Pump Co.
Jeffrey Mfg. Co.
Link-Belt Co.
Stuart Corp.
Webster Mfg., Inc.

Corrosion Protection
Amercoat Corp.
Chicago Pump Co.
Inertol Co., Inc.
Johns-Manville Sales Corp.
Koppers Co., Inc.
Perry-Austen Mfg. Co.
Stebbins Engr. & Mfg. Co.
Wallace & Tiernan Inc.

Couplings
Link-Belt Co.

Diffusers, Air (*also see* Aerators)
Alpha Ltd. (Switzerland)
American Well Works
Carborundum Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Infilco Inc.
Lakeside Engineering Corp.
Link-Belt Co.
Pacific Flush Tank Co.
Penberthy Mfg. Co., Div. of Buffalo-Eclipse Corp.
Walker Process Equipment Inc.
Wemco Div., Western Machinery Co.
Zimmer & Francescon

Diffusers, Gas (*also see* Aerators)
American Well Works
Carborundum Co.
Chicago Pump Co.
Eimco Corp.
Infilco Inc.
Walker Process Equipment Inc.
Yeomans Brothers Co.

Digestion Tank Equipment
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Gorman-Rupp Co.
Hardinge Co., Inc.
Infilco Inc.
Jeffrey Mfg. Co.
Lakeside Engineering Corp.
Link-Belt Co.
Pacific Flush Tank Co.
Smith & Loveless, Inc.
Vapor Recovery Systems Co.
Walker Process Equipment Inc.
Yeomans Brothers Co.
Zimmer & Francescon

Distributors, Rotary
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Dorr-Oliver Inc.
Eimco Corp.
Infilco Inc.
Lakeside Engineering Corp.
Link-Belt Co.
Pacific Flush Tank Co.
Smith & Loveless, Inc.
Walker Process Equipment Inc.
Yeomans Brothers Co.
Zimmer & Francescon

Dryers
Alpha Ltd. (Switzerland)
Combustion Engineering, Inc.
Eimco Corp.
Hardinge Co., Inc.
Komline-Sanderson Engr. Corp.
Link-Belt Co.
Nichols Engr. & Research Corp.
Stuart Corp.

Ejectors
Komline-Sanderson Engr. Corp.
Smith & Loveless, Inc.
Tex-Vit Supply Co.
Yeomans Brothers Co.

Engineers (*see* Directory of Engineers)

Engines (Sludge Gas and Petroleum Fuels)
Climax Engine Mfg. Co.
Fairbanks, Morse & Co.
Waukesha Motor Co.
Worthington Corp.

Fans (*also see* Air Compressors, Blowers, and Compressors)
Chicago Pump Co.
Lakeside Engineering Corp.
Westinghouse Electric Corp.

Filter Equipment, Trickling
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Dorr-Oliver Inc.
Dow Chemical Co.
Eimco Corp.
Filtration Equipment Corp.
Industrial Materials Co.
Infilco Inc.
Keasbey & Mattison Co.
Komline-Sanderson Engr. Corp.
Lakeside Engineering Corp.
Nichols Engr. & Research Corp.
Pacific Flush Tank Co.
Smith & Loveless, Inc.
Trickling Filter Floor Inst.
Walker Process Equipment Inc.
Yeomans Brothers Co.
Zimmer & Francescon

Filter Materials
Carborundum Co.
Filtration Equipment Corp.
Graver Water Conditioning Co.
Industrial Materials Co.
Johns-Manville Sales Corp.
Vulcan Materials Co.

Filters, Diatomaceous Earth
Graver Water Conditioning Co.
Yeomans Brothers Co.

Filters, Vacuum (*also see* Vacuum Filters)
Bird Machine Co.
Dorr-Oliver Inc.
Eimco Corp.
Proportioneers Div., B-I-F Industries

Flocculating Equipment
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Chain Belt Co.
Dorr-Oliver Inc.
Eimco Corp.
Graver Water Conditioning Co.
Hardinge Co., Inc.
Infilco Inc.
Jeffrey Mfg. Co.
Lakeside Engineering Corp.
Link-Belt Co.
Stuart Corp.
Walker Process Equipment Inc.
Webster Mfg., Inc.
Zimmer & Francescon

Flotation Equipment
Chain Belt Co.
Dorr-Oliver Inc.
Eimco Corp.
Graver Water Conditioning Co.
Komline-Sanderson Engr. Corp.
Yeomans Brothers Co.

Flow Measurement
Bailey Meter Co.
Burgess-Manning Co., Penn Instruments Div.
Filtration Equipment Corp.

Gas Control Equipment
Alpha Ltd. (Switzerland)
Burgess-Manning Co., Penn Instruments Div.
Ralph B. Carter Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Foxboro Co.
Homestead Valve Mfg. Co.
Pacific Flush Tank Co.
Rockwell Mfg. Co.
Vapor Recovery Systems Co.
Walker Process Equipment Inc.
Zimmer & Francescon

Gas Diffusers (*see* Diffusers, Gas)

Gas Holders, Boilers, and Appurtenances
Ralph B. Carter Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Johns-Manville Sales Corp.
Pittsburgh-Des Moines Steel Co.
Walker Process Equipment Inc.

Gaskets
Johns-Manville Sales Corp.
Keasbey & Mattison Co.

Gates
Armco Drainage & Metal Products, Inc.
Filtration Equipment Corp.
Industrial Materials Co.
Snow Gates & Valves, Inc.
Webster Mfg., Inc.

Generators
Climax Engine Mfg. Co.
Fairbanks, Morse & Co.
General Electric Co.
Westinghouse Electric Corp.
Worthington Corp.

Grease Control Materials
Cloroben Chemical Corp.

Grinders (*also see* Shredders and Grinders)
Alpha Ltd. (Switzerland)
American Well Works
Chain Belt Co.
Dorr-Oliver Inc.
Gruendler Crusher & Pulverizer Co.
Jeffrey Mfg. Co.
Yeomans Brothers Co.

Grit Collection and Conditioning Equipment
Alpha Ltd. (Switzerland)
American Well Works
Chain Belt Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Infilco Inc.
Jeffrey Mfg. Co.
Link-Belt Co.
Stuart Corp.
Walker Process Equipment Inc.
Webster Mfg., Inc.
Zimmer & Francescon

The papers given at the convention covered a wide field. The Institute was fortunate in having as the author of one of these papers Mr. J. McNicholas of Manchester, England. He reviewed what is being done in England in sewage treatment.

Sewage oxidation ponds were reviewed in two papers: one by D. R. Stanley of Edmonton, and the other by John Neil of the Ontario Water Resources Commission, Toronto.

The Humber Sewage Treatment Plant for Metropolitan Toronto received a good deal of attention through a paper by Commissioner Ross L. Clark, and by a visit to the plant.

The organic type of industrial wastes is a major problem in many centers. The treatment of these wastes was covered in a paper by L. F. Oeming of the Michigan Water Resources Commission.

Total oxidation for use in small sewage treatment plants was described in a paper by S. G. Chipman in which reference was made to a plant under construction near London, Ont.

The final paper was the result of a combination of efforts by Hans Becker, J. G. Henry, and J. C. F. MacDonald who reviewed peculiar problems encountered in the construction of trunk sewers and treatment works at Port Arthur, Ont.

At the Annual Banquet, more than 500 members and guests saw FSIWA Past-President K. S. Watson present to W. S. McFaul of Hamilton, Ont., the Bedell Award for outstanding personal service to the Canadian Institute.

At the business meeting these officers were elected to serve during 1959–60:

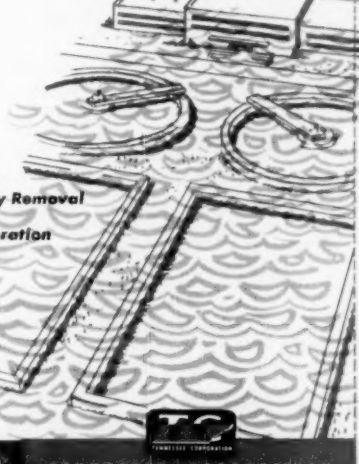
President: W. L. Waldrop, Winnipeg.
Vice-President: R. L. Clark, Toronto.
Secretary-Treasurer: A. E. Berry, Toronto.

A. E. Berry
Secretary-Treasurer

(*Continued on page 83a*)

ELIMINATE YOUR WATER TREATMENT PROBLEMS WITH . . .

ferri-floc
FERRIC SULFATE



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Excellent Taste and Odor Control
Increased Filter Runs
Coagulation Over Wide pH Range
Rapid Floc Formation *Turbidity Removal*
Manganese and Silical Removal
Bacteria Removal *Ease of Operation*
Efficient and Economical

SODIUM SILICO FLUORIDE—for Fluoridation of municipal water supplies. Available for prompt shipment. Inquiries invited.

FREE BOOKLET Let us send you without charge, a 38-page booklet that deals specifically with all phases of coagulation—just send us a postal card.

WATER TREATMENT
Efficient Coagulation of Surface or Well Water—Effective in lime soda-ash softening—Adaptable in treatment of all industrial applications.

SEWAGE TREATMENT
Coagulates over wide pH ranges and provides efficient operation regardless of rapid variations of raw sewage—Very effective for conditioning sludge prior to vacuum filtration or drying on beds.

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615/617 GRANT BUILDING, ATLANTA 3, GEORGIA

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Heating Equipment for Digesters and Buildings
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Ralph B. Carter Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
General Electric Co.
Infilco Inc.
Link-Belt Co.
Pacific Flush Tank Co.
Walker Process Equipment Inc.

Incinerators
Combustion Engineering, Inc.
Morse Boulger, Inc.
Nichols Engr. & Research Corp.
Pittsburgh-Des Moines Steel Co.
Walker Process Equipment Inc.

Ion-Exchange Equipment
Permutit Co., Div. of Pfaudler-Permutit Inc.

Insect Control
Cloroben Chemical Corp.
Leeco Chemical Co., Div. of Leeco Gas & Oil Co.

Inspection, Sewers
Centriline Corp.
Industrial Pipe Repair Corp.
National Water Main Cleaning Co.

Instruments, Recording and Control (*also see* Controls and Recorders)
Bailey Meter Co.
Builders-Providence Div., B-I-F Industries
Burgess-Manning Co., Penn Instruments Div.
Fischer & Porter Co.
Foxboro Co.
General Electric Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
Rockwell Mfg. Co.
Wallace & Tiernan Inc.
Westinghouse Electric Corp.

Jointing Materials
American Concrete Pressure Pipe Assn.
Gray Concrete Pipe Co., Inc.
Keasbey & Mattison Co.
National Clay Pipe Mfrs., Inc.
Perry-Austen Mfg. Co.
Stebbins Engr. & Mfg. Co.
Wedge-Lock Clay Pipe Mfrs., Inc.

Joints, Mechanical
American Cast Iron Pipe Co.
Johns-Manville Sales Corp.
Smith-Blair, Inc.
U. S. Pipe & Foundry Co.
Wedge-Lock Clay Pipe Mfrs., Inc.

Laboratory Equipment and Supplies
Filtration Equipment Corp.
General Chemical Div., Allied Chemical Corp.
Stuart Corp.

Lift Stations
Smith & Loveless, Inc.
Tex-Vit Supply Co.

Lighting Fixtures
Westinghouse Electric Corp.

Lubricants
Homestead Valve Mfg. Co.
Johns-Manville Sales Corp.

Lumber
Vulcan Materials Co.

Manhole and Inlet Castings
American-Marietta Co.
Vapor Recovery Systems Co.
Vulcan Materials Co.

Masonry Building Materials
American-Marietta Co.
Portland Cement Assn.
Price Bros. Co.
Vulcan Materials Co.

Meter Boxes
Johns-Manville Sales Corp.

Meters (Sewage, Sludge, Water, Air, and Gas)
Bailey Meter Co.
Builders-Providence Div., B-I-F Industries
Burgess-Manning Co., Penn Instruments Div.
Ralph B. Carter Co.
Filtration Equipment Corp.
Foxboro Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
Rockwell Mfg. Co.
Roots-Connersville Blower, Div. of Dresser Industries Inc.
Simplex Valve and Meter Co.
Worthington Corp.

Mixing Devices
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Chain Belt Co.
Dorr-Oliver Inc.
Eimco Corp.
Infilco Inc.
Jeffrey Mfg. Co.
Walker Process Equipment Inc.
Wemco Div., Western Machinery Co.

Motors
Chicago Pump Co.
Fairbanks, Morse & Co.
General Electric Co.
Link-Belt Co.
Westinghouse Electric Corp.
Worthington Corp.

Odor Control Materials
Airkem Inc.
Cloroben Chemical Corp.
Florasynth Laboratories, Inc.
Tennessee Corp.
Wallace & Tiernan Inc.

Odor Counteractants
Airkem Inc.

Package Treatment
Chain Belt Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Infilco Inc.
Municipal Service Co.
Smith & Loveless, Inc.
Yeomans Brothers Co.

Packing
Johns-Manville Sales Corp.
Keasbey & Mattison Co.

Paints and Protective Coatings
Amercoat Corp.
Inertol Co., Inc.
Koppers Co., Inc.
Perry-Austen Mfg. Co.
Pittsburgh Coke & Chemical Co.
Smith & Loveless, Inc.
Stebbins Engr. & Mfg. Co.

Pipe, Asbestos Cement
Industrial Materials Co.
Johns-Manville Sales Corp.
Keasbey & Mattison Co.

Pipe, Cast Iron
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
Industrial Materials Co.
U. S. Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Clay
Industrial Materials Co.
National Clay Pipe Mfrs., Inc.
Wedge-Lock Clay Pipe Mfrs., Inc.

Pipe Cleaning
Centriline Corp.
Cloroben Chemical Corp.
Flexible Inc.
Homestead Valve Mfg. Co.
National Water Main Cleaning Co.
W. H. Stewart, Inc.

Pipe Coatings and Linings
Amercoat Corp.
American Pipe & Construction Co.
Centriline Corp.
Koppers Co., Inc.
National Water Main Cleaning Co.
Perry-Austen Mfg. Co.
Stebbins Engr. & Mfg. Co.
Vulcan Materials Co.

Pipe, Concrete
American Concrete Pressure Pipe Assn.
American-Marietta Co.
American Pipe & Construction Co.
Gray Concrete Pipe Co., Inc.
Lock Joint Pipe Co.
Portland Cement Assn.
Price Bros. Co.
Vulcan Materials Co.

Pipe, Fiber
Brown Co.

Pipe Fittings
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
Keasbey & Mattison Co.
Price Bros. Co.
Smith-Blair, Inc.
U. S. Pipe & Foundry Co.
R. D. Wood Co.

Pipe Jointing Materials (*see* Jointing Materials)

Pipe, Plastic
Amercoat Corp.

Pipe, Repairs
Centriline Corp.
Industrial Pipe Repair Corp.
Smith-Blair, Inc.

Pipe, Steel
American Pipe & Construction Co.
Armco Drainage & Metal Products, Inc.
Keasbey & Mattison Co.

Plastic Pipe Products
Amercoat Corp.
Keasbey & Mattison Co.

Pump Controls
Builders-Providence Div., B-I-F Industries
Burgess-Manning Co., Penn Instruments Div.
Chicago Pump Co.
Fischer & Porter Co.
Foxboro Co.
General Electric Co.
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
Rockwell Mfg. Co.
Smith & Loveless, Inc.
Tex-Vit Supply Co.
Westinghouse Electric Corp.
Worthington Corp.
Zimmer & Francescon

KENTUCKY–TENNESSEE INDUSTRIAL WASTES AND SEWAGE ASSOCIATION

The 13th Annual Meeting of the Kentucky-Tennessee Industrial Wastes and Sewage Works Association was held at the Lafayette Hotel, Lexington, Ky., Sept. 14–16, 1959. Registration totaled 316, including ladies. The meeting was held jointly with the Kentucky-Tennessee Section of the American Water Works Association.

Among the topics discussed were:

"Organization, Aims and Progress of Kentucky Water Resources Commission," by S. Russell Marshall, City Manager, Frankfort, Ky.

"Cost of Relocation of Utilities," by Edwin C. Hoskins, Knoxville Utilities Board, Knoxville, Tenn.

"Radioactivity in Streams," by Harry P. Cramer, U. S. Public Health Service, Cincinnati, Ohio.

"Oil Pollution Problems in Green County, Kentucky," by Walter C. Martin and H. D. Tomlinson, Kentucky Water Pollution Control Commission, Louisville, Ky.

"Operational Problems of New Sewage Treatment Plants in Tennessee and Kentucky"—Panel Discussion.

"Sludge Thickeners," by W. E. Budd, Dorr-Oliver Inc., Stamford, Conn.

"New Developments in Mechanical Aeration and Activated Sludge Treatment," by A. A. Kalinske, Infilco, Inc., Tucson, Ariz.

"Total Aeration for Domestic and Industrial Wastes," by Edward C. Rubin, Chicago Pump Co., Chicago, Ill.; Robert K. Barefoot, Dravo Corp., Cleveland, Ohio; and R. C. Cloppen, Yeomans Brothers Company, Melrose Park, Ill.

A field trip to the industrial wastes treatment facilities of the IBM plant at Lexington was a special event of the last day.

At the banquet, FSIWA Executive Secretary Ralph E. Fuhrman presented the Hatfield Award for outstanding treatment plant operation and reporting to W. T. Blackman, Sr., Superintendent, Murfreesboro, Tenn. The Bedell Award for extraordinary service to the Association went to S. Leary Jones, Director, Stream Pollution Control Board, Nashville, Tenn.

The following officers were elected:

Chairman: A. L. Strub, Union City, Tenn.

Vice-Chairman: C. M. Fair, Louisville, Ky.

Secretary-Treasurer: S. Leary Jones, Nashville, Tenn.

S. Leary Jones
Secretary-Treasurer

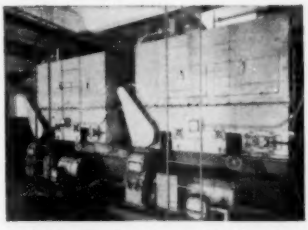
SOUTH DAKOTA WATER AND SEWAGE WORKS CONFERENCE

The 25th Annual Meeting of the South Dakota Water and Sewage

(*Continued on page 85a*)

—Manufacturers Since 1885—

Gruendler's Success in Brooklyn, New York



View of TWO Gruendler Sewage Shredders installed in connection with Bar Screens in the Coney Island Sewage Treatment Plant of Brooklyn, New York.

The Gruendler Multiple Unit Shredder installations here have solved a very difficult problem of sewage disposal. Sticks, rags and other debris are thoroughly reduced to a fine consistency for an even and continuous flow through their system. Two of the four Shredder Units installed in Brooklyn have been in operation for ten years. Hundreds of Gruendler Shredder installations in cities all over the Nation testify to their quality, precision features and efficient operation.

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Pumps, Airlift
Walker Process Equipment Inc.

Pumps, Chemical Feed
Proportioneers Div., B-I-F Industries

Pumps, Gas
Roots-Connersville Blower, Div. of Dresser Industries Inc.

Pumps, Grit
Nagle Pumps, Inc.

Pumps, Sewage
Alpha Ltd. (Switzerland)
American Well Works
Chicago Pump Co.
Eimco Corp.
Fairbanks, Morse & Co.
Gorman-Rupp Co.
Infilco Inc.
Marlow Pumps, Div. of Bell & Gossett Co.
Smith & Loveless, Inc.
Wallace & Tiernan Inc.
Wemco Div., Western Machinery Co.
Worthington Corp.
Yeomans Brothers Co.
Zimmer & Francescon

Pumps, Sludge
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
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Works Conference opened at Huron, S. Dak., on Sept. 2, 1959, with a smoker at the Marvin-Hughitt Hotel. Registration for the meeting totaled 166.

The technical program started with a talk on the history of the Conference by W. W. Towne, U. S. Public Health Service. This was followed by a discussion of "Federation of Sewage and Industrial Wastes Associations—Past, Present, and Future," by Kenneth W. Watson, Past-President of the Federation.

During the afternoon "Radioactive Wastes—A New Problem" was discussed by Everett R. Mathews, Chief, Health Protection Branch, U. S. Atomic Energy Commission.

The Annual Banquet was held in the Marvin-Hughitt Hotel with Mayor R. B. Matson of Huron as toastmaster. The Izaak Walton League Award was presented to Frank Thomas, Sewage Treatment Plant Operator of Custer. Flox Company Awards were presented to Erwin Wollman of Freeman, and to James Cox of Sioux Falls.

A panel discussion on "Developments in Water and Sewage Works Equipment and Material" was the main feature of Friday morning. Those who participated in the discussion were: Charles E. Carl, South Dakota Department of Health; Willard J. Bell, Wallace & Tiernan, Inc.; Mel Bischoff, Flox Company; Fred Voedisch, Layne-Minnesota Company; M. O. Thomas, Lyons Chemical, Inc.; and William V. Campbell, Campbells, Inc.

The remainder of the morning was devoted to the annual business meeting.

Officers for the forthcoming year were presented at the annual business luncheon. They were:

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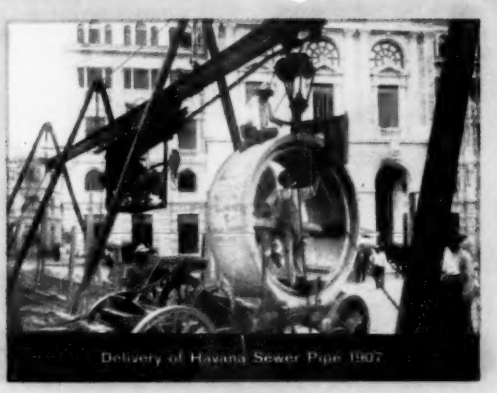

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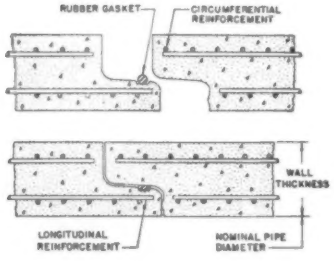

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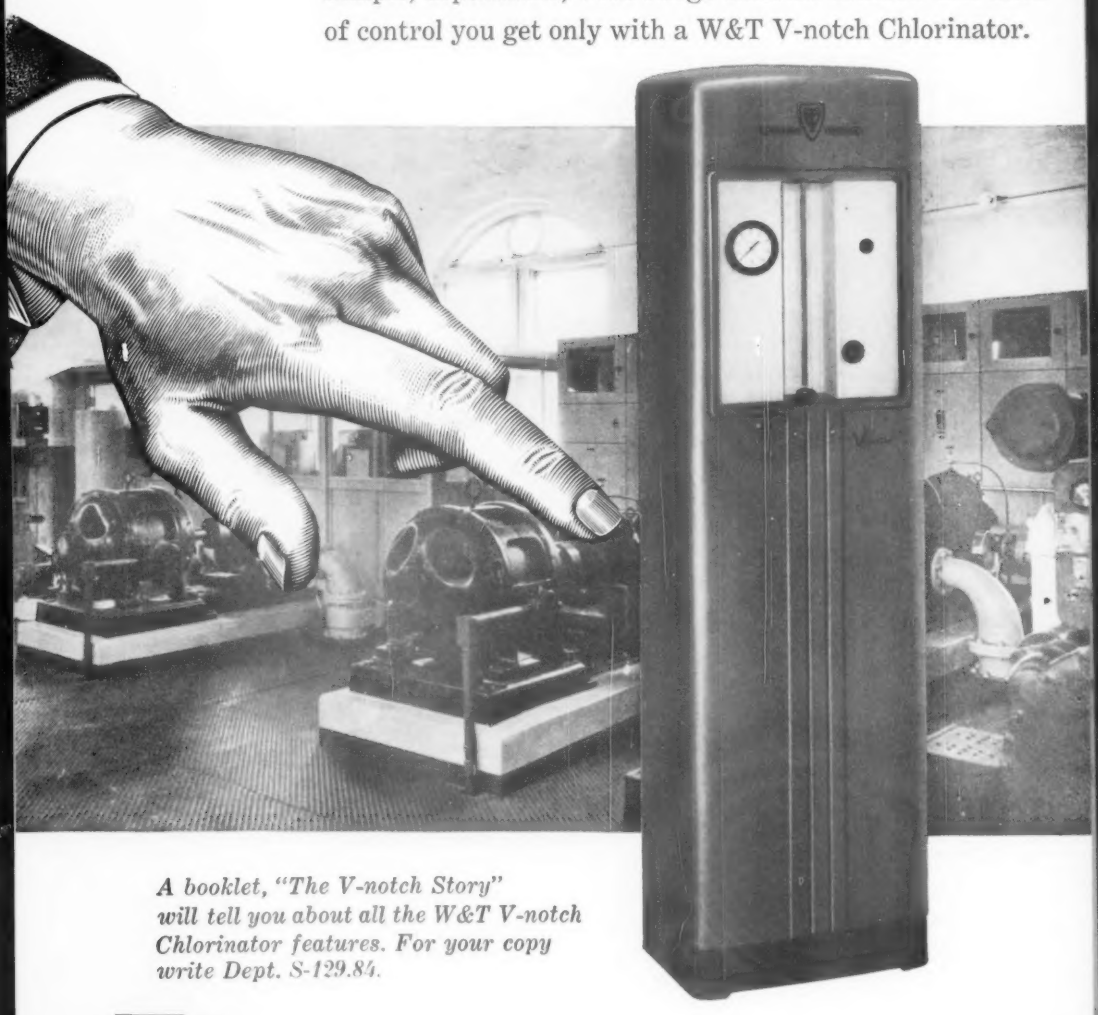

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